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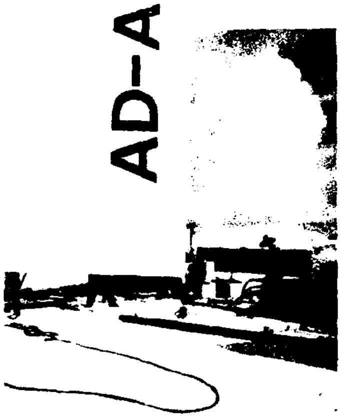

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FOUNDATION INVESTIGATION FOR GROUND BASED RADAR PROJECT--KWAJALEIN ISLAND, MARSHALL ISLANDS

by

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Results of a foundation investigation for the Ground Based Radar Project -- Kwajalein Island, Marshall Islands, are presented. Geophysical tests comprised of surface refraction and borehole seismic tests were performed. Results from geotechnical in situ tests, which included standard penetration and plate bearing tests, are also presented to provide a comprehensive report on the foundation materials. The elastic properties, Young's modulus, Poisson's ratio, shear modulus, and constrained modulus were determined as a function of depth for the foundation soils. These properties are necessary to design a stable foundation that will experience dynamic loading.

Foundation materials encountered at this site were coral sands, silts, and gravels to a depth of 60 ft (el -52). The upper 14 ft of foundation soils are a hydraulically placed fill of beach sand. The materials are medium dense to dense sands. A global average SPT $N_1(60)$ value of 22 is appropriate for the site and correlates to a sand with a relative density of 65 percent.

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The shear modulus ranges from $7.7 \times (10^3)$ psi. Poisson's ratio varies from 0.40 for unsaturated to 0.49 for saturated soil.

A method was presented to estimate moduli under the expected load from present unloaded in situ moduli values using the K2 parameter. An estimate of the moduli for the expected foundation load was presented for the upper 30 ft of the foundation. A comparison of the plate bearing test and seismic derived Young's modulus shows good agreement, $123 \times (10^3)$ psi and $78 \times (10^3)$ psi, respectively.

Preface

This report documents the foundation investigation conducted by the US Army Engineer Waterways Experiment Station (WES) for the Ground Based Radar Project at Kwajalein Island, US Army Kwajalein Atoll, Marshall Islands. The work was performed during the period 25 October through 3 November 1989 for the Engineering Division of the US Army Engineer Division, Pacific Ocean (POD), under IAO No. E87890083 POD MIL-R dated 8 Sep 89.

Mr. George Masatsugu, Geotechnical Section, Engineering Division (ED-G), POD, was Project Monitor for this work. Also, Mr. Olson Okada, ED-G, was the onsite monitor during the field work. Mr. Brad Scully was the POD Engineering Division Coordinator, Military Project Management Section (ED-NP). Their assistance was instrumental in the successful completion of this work.

Mr. Donald E. Yule of the Earthquake Engineering and Seismology Branch (EESB), Earthquake Engineering and Geosiences Division (EEGD), Geotechnical Laboratory (GL), WES, was the Project Engineer for this study. Mr Michael K. Sharp, Engineering Geophysics Branch (EGB), EEGD, GL, was the coinvestigator and coauthor of this report. The field work was performed by Messrs. D. E. Yule and M. K. Sharp. The work was conducted under the direct supervision of Mr. J. R. Curro, Chief, EGB, Dr. Mary Ellen Hynes, Chief, EESB, and Dr. A. G. Franklin, Chief, EEGD. The project was under the overall supervision of Dr. W. F. Marcuson III, Chief, GL.

COL Larry B. Fulton, EN, was Commander and Director of WES during the investigation. Dr. Robert W. Whalin was Technical Director.

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Conversion Factors, Non-SI to SI (Metric) Units of Measurement

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	2.54	centimetres
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
slugs (mass)	14.5939	kilograms
tons (force) per square foot	95.76052	kilopascals

FOUNDATION INVESTIGATION FOR GROUND BASED RADAR PROJECT

KWAJALEIN ISLAND, MARSHALL ISLANDS

Introduction

1. Background. The foundation investigation was performed adjacent to the Defense Control Center (DCC) building which is an existing structure on which a tower will be constructed to support a radar antenna. The existing building foundation needed to be evaluated to ensure that it would provide a stable foundation during the dynamic loading of this new antenna support system. Testing within the building itself was not possible; therefore, tests were conducted near the building in an area that was included in the earth berm surcharged foundation of the existing DCC building. It is assumed that site conditions at the test locations and beneath the building are virtually the same because of the proximity and similar preloading conditions.

2. Purpose. The purpose of this investigation was to assess the foundation materials at the site to a depth of 60 ft[1] by employing in situ geophysical and geotechnical methods. Geophysical methods were used to determine compression (P)- and shear (S)- wave velocities so that a velocity zonation of the foundation materials could be determined. For this study, a

----

1. A table of factors for converting non-SI to SI (metric) units of measurement is presented on page 6.

suite of seismic methods consisting of surface refraction, downhole, and crosshole tests were conducted to determine the above values. The geotechnical methods employed Standard Penetration Testing (SPT), density determinations, gradation, classification, and indexing tests of selected soil samples. Also, a Plate Bearing Test was performed at one location at the site. The geotechnical tests were performed by POD personnel. The SPT's and laboratory sample testing provided N-values, material classification and density of the foundation materials. Knowing this information the elastic constants Young's modulus (E), Shear modulus (G), and Poisson's ratio (ν) can be determined for the foundation materials which are needed in the design of the antenna support structure. The plate bearing test provided an alternate method for an in situ determination of E.

Site Description

3. General. The location of this study was the US Army Kwajalein Atoll (USAKA) which is located in the northern Marshall Islands in the west central Pacific Ocean. The site is located at the western end of Kwajalein Island adjacent to the western side of the DCC building, as shown in figure 1. The regional geology of the site is an island derived from the buildup of coral skeletons on the submerged rim of an extinct volcano. The topography is low and flat. The elevation (el[2]) of the site is approximately 8 ft above mean

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2. All elevations are in feet and are referenced to mean sea level

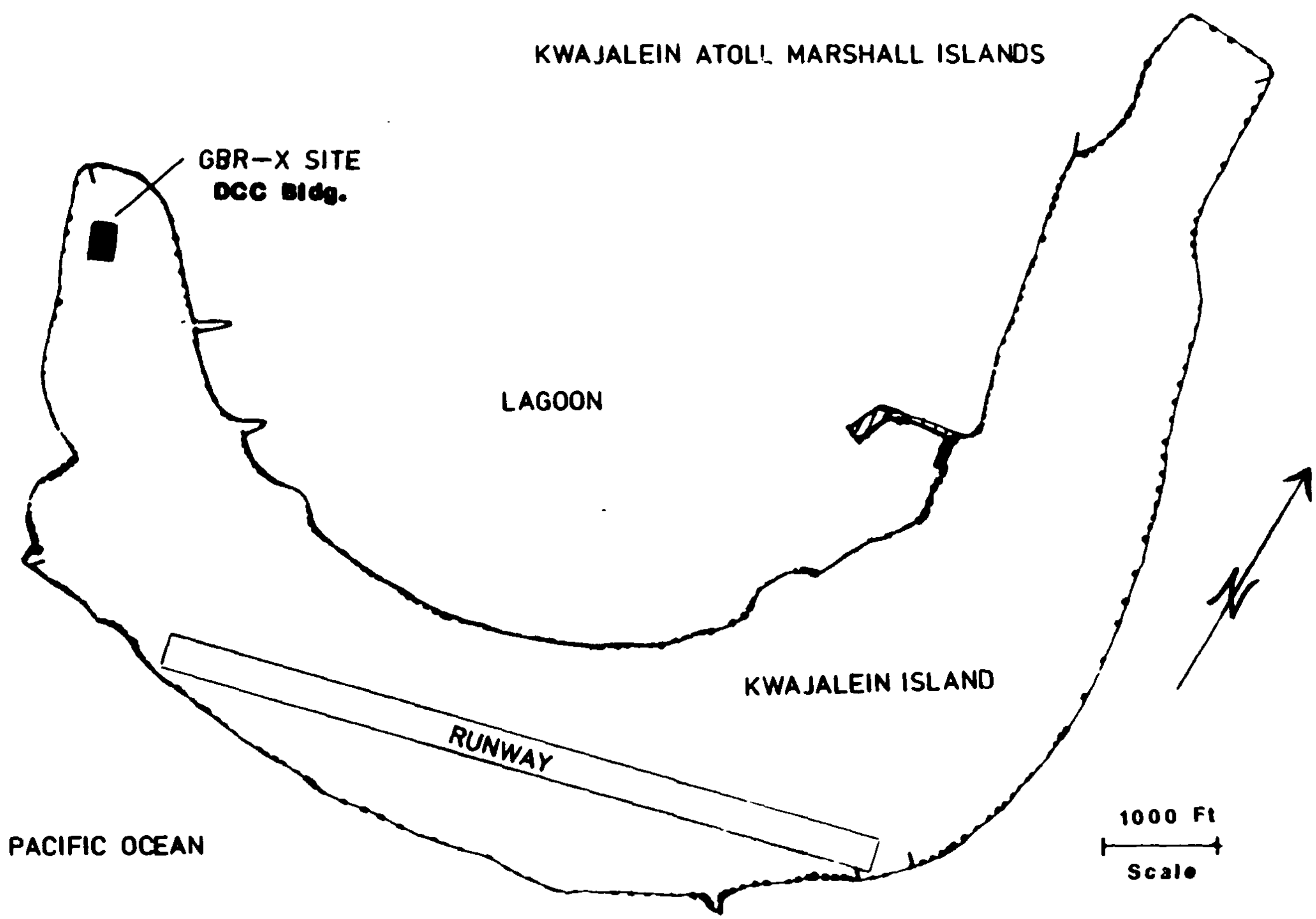


Figure 1. Site Location

sea level. The materials composing the subsurface are unconsolidated calcareous materials on a reef surface. The subsurface materials within the landmass are composed of unconsolidated limestone derived sediments of cobbles to silt sizes with a general USCS classification of SP. Also, the depth of the reef rock increases rapidly in the direction of the lagoon (US Army Engineers, 1989). This general geologic description of the site agrees well with the information derived from the borings drilled to accommodate the subsurface geophysical tests.

4. Foundation. The foundation at the site is composed of naturally occurring materials below a depth of 14 ft (el -6). These materials are composed of sands with silt and gravel with the occurrence of gravel increasing with depth. The boring logs contained no indication of coral limestone rock within the upper 60 ft. This information was obtained from the boring logs from holes BH-1, BH-2, and BH-3 which are included in Appendix A. A fill of beach sand was hydraulically placed to a height of 8 ft above the original island surface. This fill was surcharged with 8 ft of additional fill which was removed prior to start of construction. The water table is found at an average depth of 8 ft (el 0) and fluctuates ± 2 ft with the tide.

5. Idealization. For clarity and use in engineering analysis it is necessary to idealize the foundation materials into discrete layers which will then be identified by material type. Material properties are assigned to each layer based upon the test results. The geological setting and construction

history of the site suggest idealization into four layers. The top layer consisting of fill will be variable and contain a shallow near surface zone of fill that may be disturbed and contain various construction materials buried during the construction and cleanup of the site in conjunction with the construction of the DCC building. This zone would then change into undisturbed hydraulic fill placed above the water table. A change in character of the fill placed below the water table might be evidenced and would be caused by the different placement conditions. Layer two would be the naturally occurring near surface ocean deposited beach materials. The third and fourth layers would consist of the coarser and more dense unconsolidated materials.

Test Program

6. The locations of tests performed during this investigation are shown in figure 2. All phases of the geophysical test program, except the crosshole S-wave test, were conducted according to Engineering Manual 1110-1-1802 guidelines (Department of the Army, 1979). The surface portion of the test program consisted of three seismic refraction lines (R1,R2,R3) in the vicinity of the plate bearing test location. Crosshole and downhole testing were performed near the DCC building in three boreholes (BH-1,2,3) spaced 10 ft and 5 ft apart, respectively, which were drilled to a depth of 60 ft. In addition to the geophysical tests employed in the completed boreholes, SPT and sampling were performed during the drilling operation.

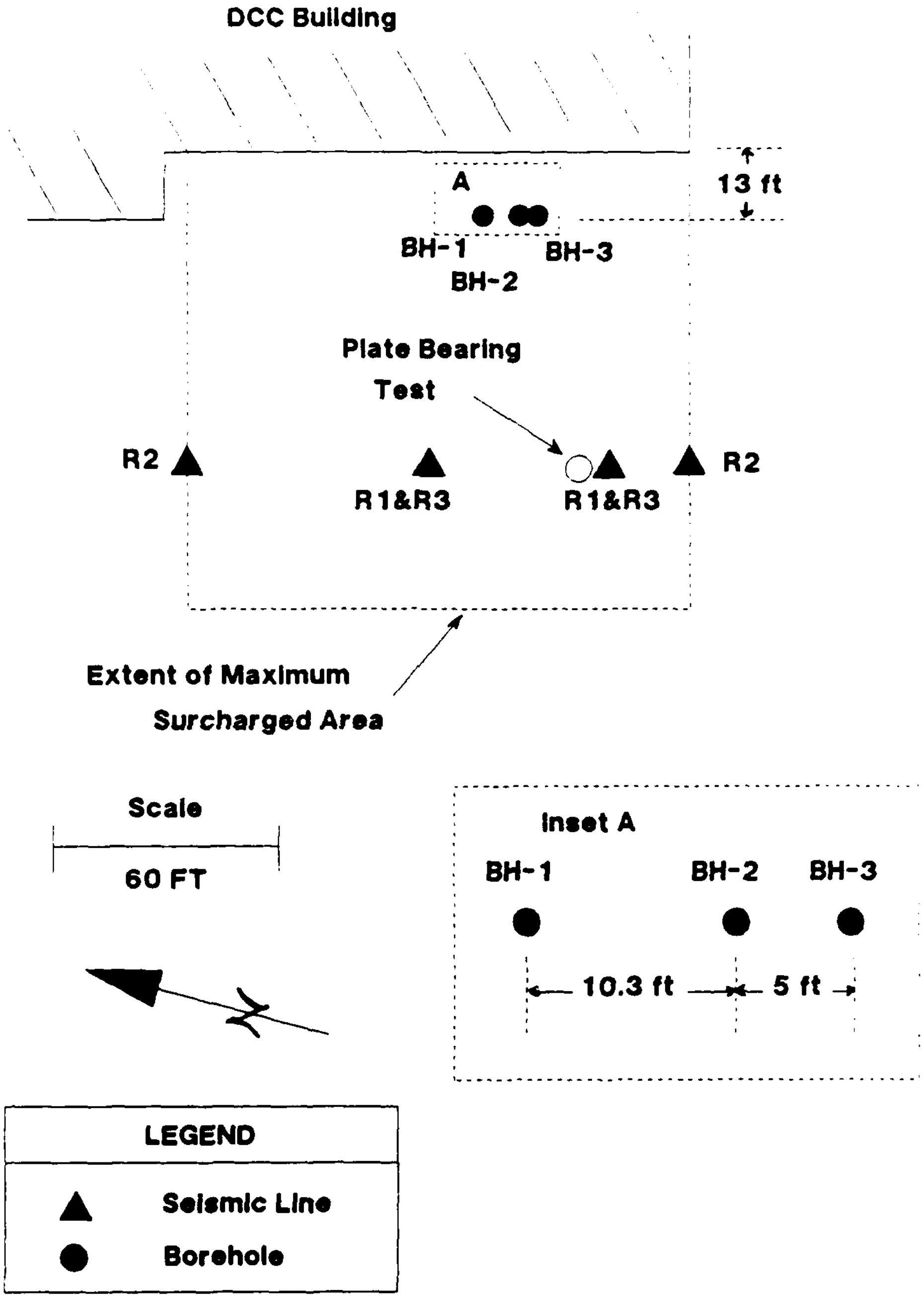


Figure 2. Test Layout

Geophysical Tests Procedures

7. Crosshole. In preparation for the crosshole testing, 8-in. diameter boreholes were drilled to the 60-ft depth. These holes were then cased with schedule 40 4-in. ID PVC pipe with the annular space between the borings and casings grouted with a material that approximates the consistency of soil when it sets. The vertical alignment of each borehole was checked to see if there was any appreciable drift since accurate reduction of the crosshole data requires that the straight-lined distance be known between source and receiver at each test elevation. The S-wave test procedure consisted of placing a downhole triaxial geophone array in the receiver hole(s) and a downhole electrically powered vibrator in the source hole. The vibrator is frequency and duration controlled and produces a repeatable and vertically polarized shear (SV) -wave which allows accurate arrival time determination. The source and receiver(s) were lowered to the same depth in a borehole set (one boring for the source and two borings for the receivers) and clamped to the casing walls using inflatable bladders or pneumatically powered rams. The vibrator frequency was varied between 50 and 500 Hz using a four cycle burst mode and monitored until an optimal frequency was found that propagated well at that depth. The source waveform and the received waveform were recorded using a digital seismograph. The data were stacked (enhanced) until a well defined waveform was produced. For the P-wave test, the seismic source was an exploding bridgewire (EBW) detonator which was sufficiently strong in energy that data stacking was not necessary. Several different test configurations

were employed in the test program that are tabulated in figure 3. Knowing the distance between the source and receiver and the P- and S- wave arrival times at each test depth, an analysis of these data sets was made with the aid of a computer program "CROSSHOLE" (Butler et al, 1978). This program calculates true P- and S-wave velocities and determines velocity zones and depths to interfaces.

8. Downhole. The downhole test is similar to the crosshole test except the source is kept at the surface while the receiver array is lowered in a boring at 5-ft intervals. The source for the S-wave test is a hammer striking a wooden plank on alternate ends, which produces two records. The seismic signals produced by this procedure are predominantly horizontally polarized S-waves, with polarity depending on the direction of the hammer strike. The signals detected by the horizontal geophones on these two records are overlain and examined for a polarity reversal which is considered the arrival of the S-wave. The P-wave source for this test was a downward hammer blow to a steel plate with the vertical geophone signal being used to determine the P-wave arrival. The data are reduced by plotting arrival times versus slant distance between source and receiver. The inverse slope of the line segments drawn through the data points gives the velocities and slope changes in the line segments indicate the approximate depths where the velocities change. The various test configurations used in this study are shown in figure 3.

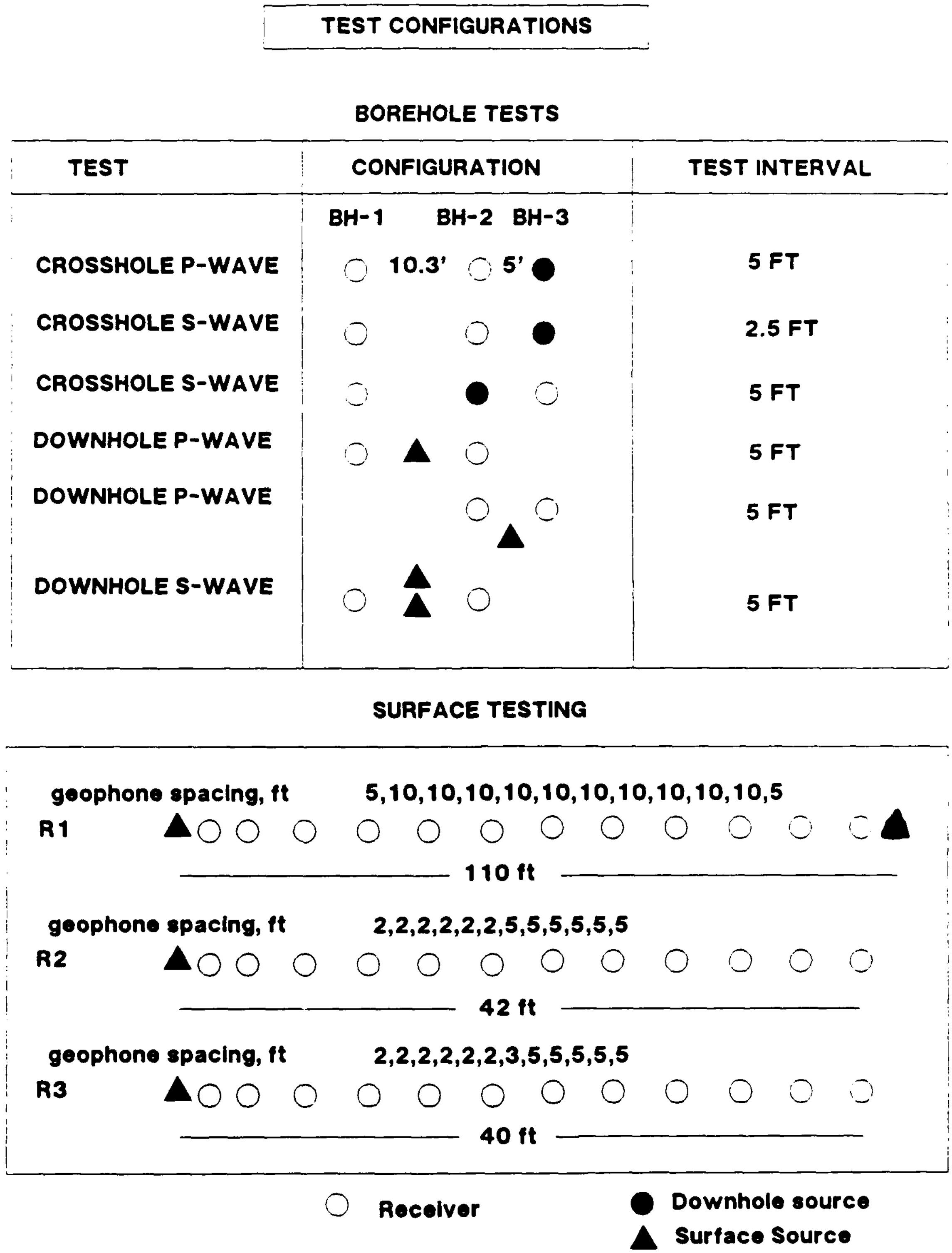


Figure 3. Test Configurations

9. Surface seismic refraction. The procedure for the surface refraction tests was to place geophones along a straight line on the ground surface at selected intervals with an energy source initiated at a selected offset distance from each end of the geophone array. Two types of surface tests were performed which consisted of a P-wave and S-wave test. Lines R1 and R2 were P-wave tests and line R3 was a S-wave test. The energy sources for these tests were identical to the ones used for the downhole P- and S-wave tests. The test configuration for each line is shown in figure 3. The data reduction consisted of plotting first arrival time of the P- and S- wave signal detected at each geophone versus geophone distance from the source. From these time versus distance (TD) plots, velocities and depths to refracting interfaces were determined using the computer program "SEISMO" (Yule and Sharp, 1989).

Geotechnical Test Results

10. Soil classification and density. Analysis of the soil sampling data has produced a four layer profile interpretation which is illustrated in figure 4. The first layer of fill material is a medium dense to dense poorly sorted sand (SP) with zones of silty sand and some cobbles and gravel. This layer extends to a depth of 13 ft. The second layer begins with the original soil deposit. This layer is a medium dense silty, very fine grained sand (SM) which can be found between 14 and 19 ft in depth (el -6 to -11). The third

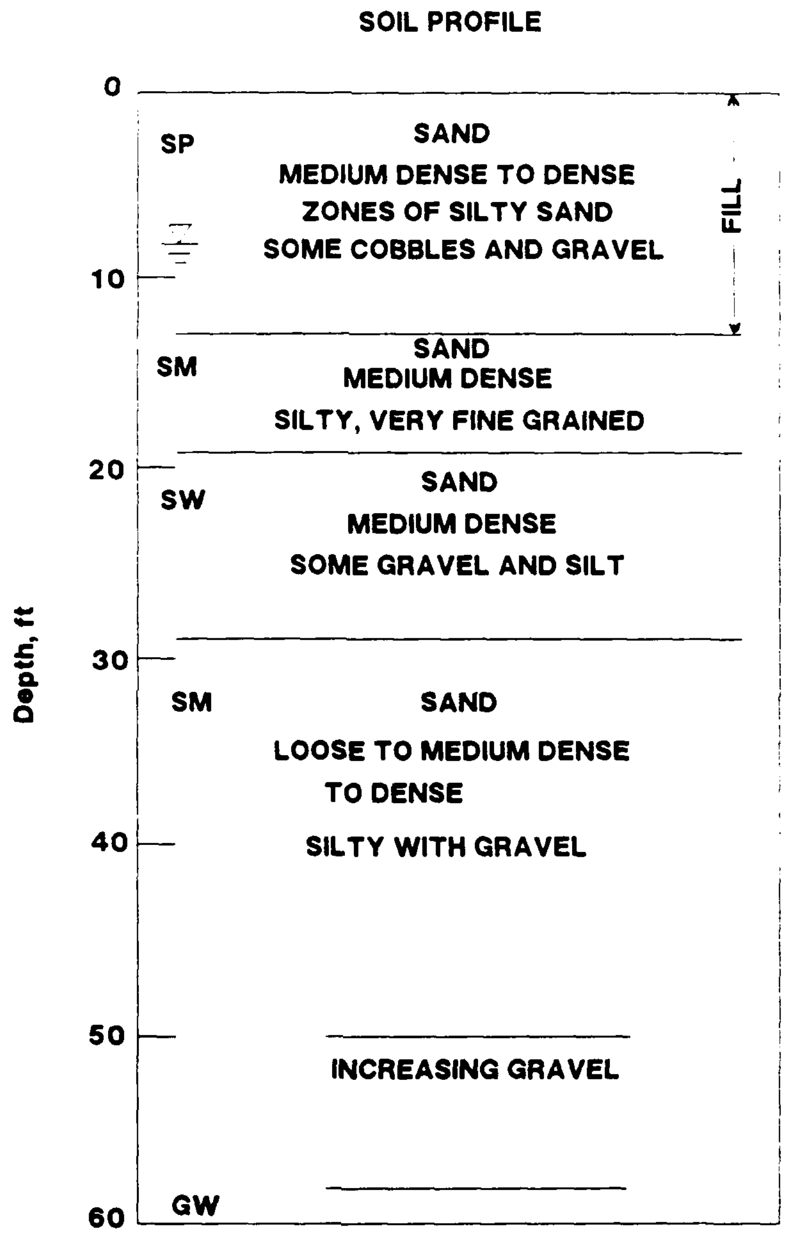


Figure 4. Soil Profile

layer is a medium dense well graded sand with some silt and gravel that ends at a depth of 29 ft (el -21). The fourth layer is a loose to medium dense to dense silty sand with an increasing gravel content below 50 ft (el -42).

11. Density determinations of samples above the water table in the fill layer show an average moist unit weight of 100 lb/ft^3 and a water content of 11 percent. The specific gravity of the solids was measured to be 2.8. These results were used to estimate a total unit weight of the soil below the water table of 120 lb/ft^3. Detailed information of the testing program and individual sample test results can be found in the boring logs, Appendix A.

12. SPT. The SPT's performed in boreholes BH-1,2,and 3 are presented in figure 5 as two plots of N-values versus depth. The recorded SPT data can be found in the boring logs, Appendix A. The leftmost plot presents the raw or measured blowcounts. In the next plot, the $(N1)_{60}$ value corresponding to each measured N-value is shown. The $(N1)_{60}$ value is determined by adjusting the observed N-value to an equivalent N-value had the test been conducted at a vertical effective overburden stress of 1 ton/ft^2 at an energy level of 60 percent of the theoretical maximum applied energy of the drop weight. The SPT's from this project were performed using the rope and cat-head system which operates at the 60 percent energy level. Determining an equivalent $(N1)_{60}$ blowcount allows the results to be compared with other equivalent $(N1)_{60}$ values and, therefore, established correlations of relative density can

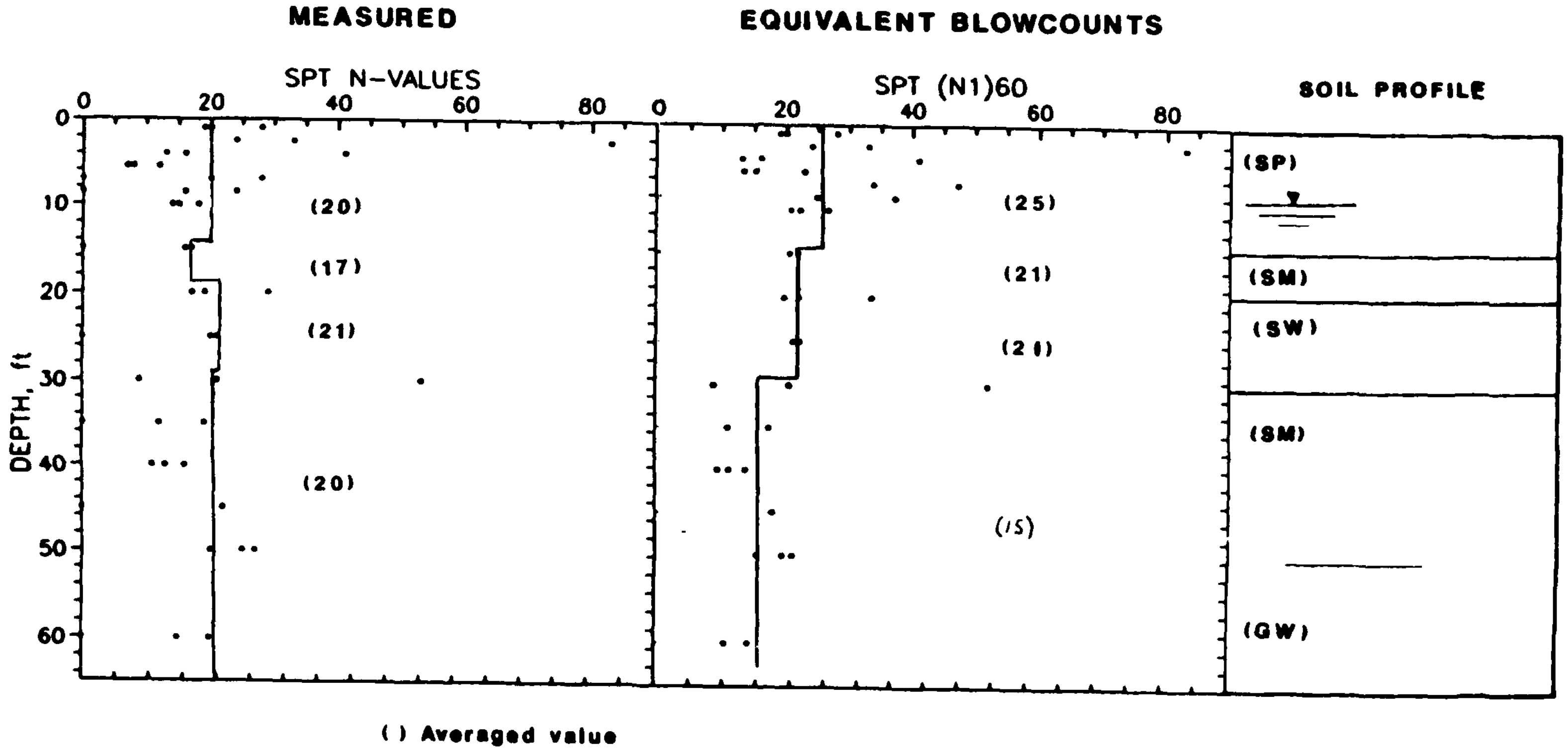


Figure 5. Standard Penetration Test Results

be used to estimate these parameters for the soil at the site. The $(N1)_{60}$ was obtained by multiplying the measured blow counts by the correction factor Cn, which was determined from the curves presented in figure 6. The vertical effective stress was calculated using a total unit weight of 100 lb/ft^3 for the soil above the water table and a saturated unit weight of 120 lb/ft^3 for soil below the water table. A depth to the water table of 8 ft was used in these calculations.

13. The SPT data show considerable scatter especially near the surface. The average measured and average equivalent $(N1)_{60}$ blowcounts are annotated in figure 5 and in table 1 for each soil layer. These representative values for each layer are plotted as vertical lines on the plots. Using the $(N1)_{60}$ values and the correlation curve for sands given in figure 7, estimates of the relative density of the sands at the site were determined and listed in table 1. A site global average $(N1)_{60}$ value of 22 is appropriate for the sands and correlates to sand with a relative density of 65 percent.

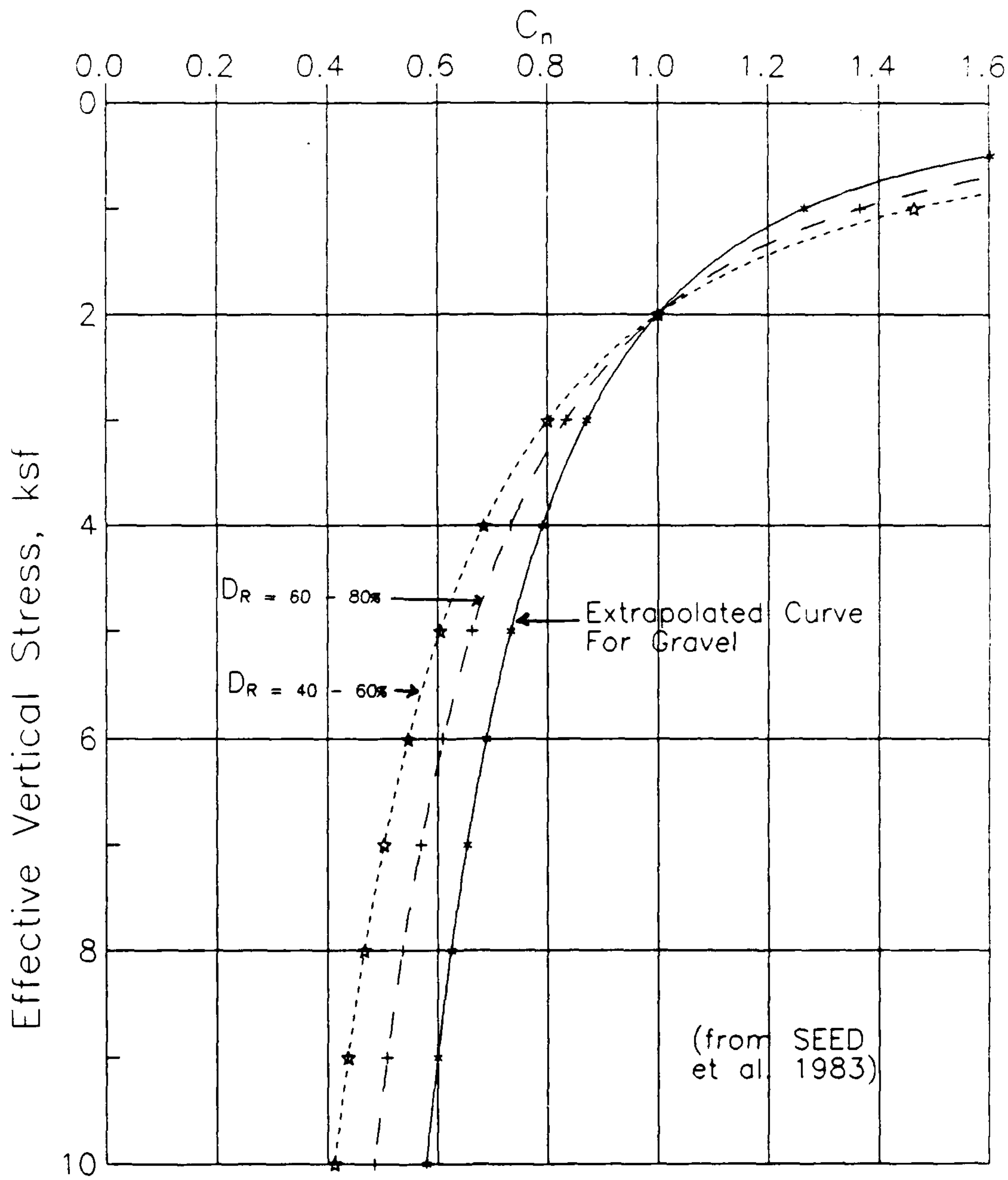


Figure 6. C_n Curves Used to Convert N to N_1

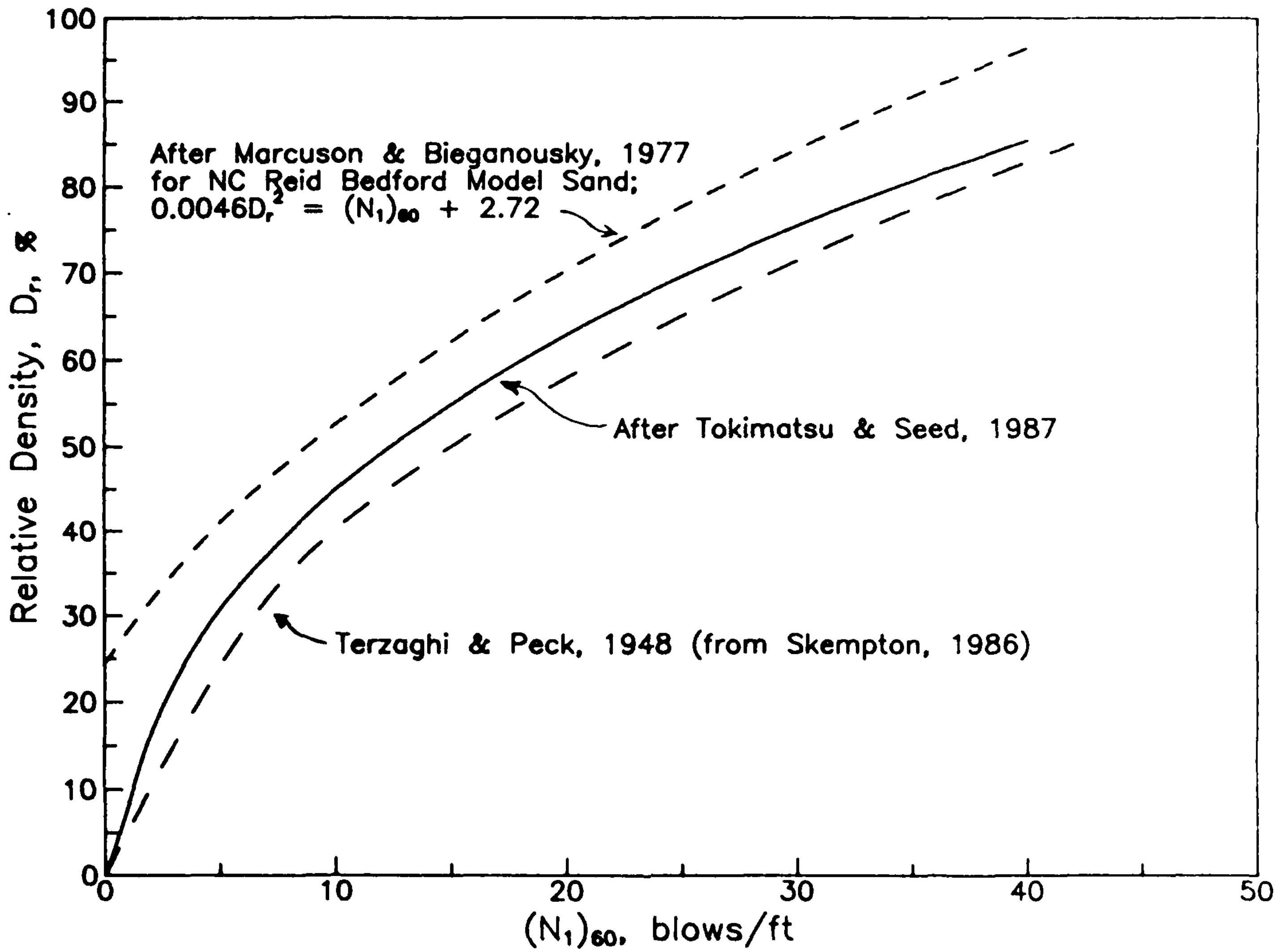


Figure 7. Relationship Between Relative Density and $(N1)_{60}$ of Sands

Table 1. SPT Test Results

Layer	Average		Relative Density, %
	N(measured)	$(N1)_{60}$	(based on figure 7) Tokimatsu & Seed 1987)
1	20	25	67
2	17	21	64
3	21	21	64
4	20	15	55

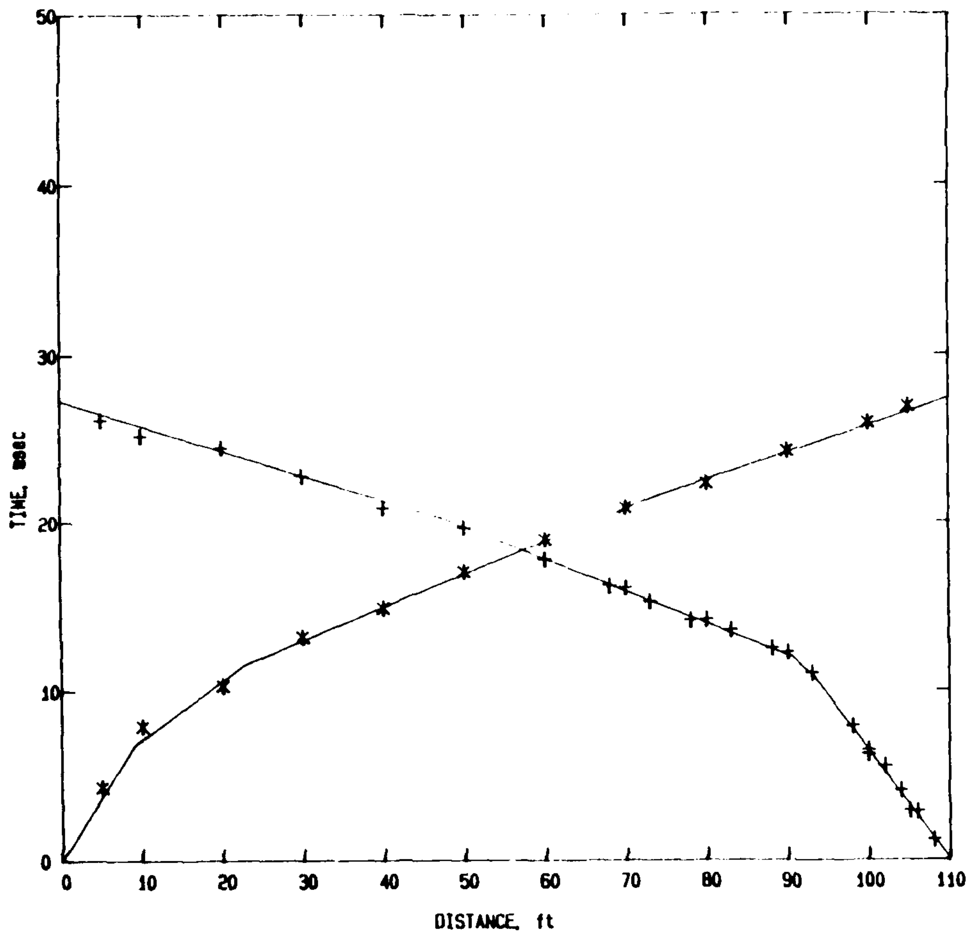
14. Plate Bearing Test. A Plate Bearing Test was conducted at one location and consisted of static and cyclic [illegible] tests on a 26.6- in. square steel plate. The testing elevation was 1 ft below the existing grade (el 7). The complete description of these tests and their results are presented in Appendix B. For the static plate bearing test, a maximum load of 34.9 lb/in.^{2} was applied. Based upon this test, a modulus of soil reaction of 340 lb/in.^{3} was determined. The cyclic plate bearing test was performed with a 19.3 lb/in.^{2} static load and a cyclic load of 5 lb/in.^{2}. From this test an E of 123.6x(10^3) lb/in.^{2} was determined. This computation is presented in Appendix B.

Geophysical Test Results

Surface Seismic Refraction

15. P-wave Tests R1 and R2. The results from P-wave surface refraction lines R1 and R2 are shown in figure 8. The R2 line was configured to investigate the near-surface materials; whereas, the longer R1 line was employed to investigate to deeper strata. The two overlapping data sets were combined to provide a detailed interpretation. From this composite TD plot a four layer seismic profile was indicated. A cross section is also shown in figure 8 with the depths to interfaces and velocities presented. The calculated depths at each end of the line indicate very little dipping of the layers in this area. Layers one and two with velocities of 1,440 ft/sec and 2,610 ft/sec, respectively, correspond to the fill materials above the water table. The third layer with a velocity of 5,110 ft/sec is the saturated fill material. This result shows the "seismic" water table to be at an average depth of 8.0 ft (el 0). The fourth layer detected at a depth of 17-19 ft (el -9 to -11) has a velocity of 6,400 ft/sec and corresponds to the saturated well sorted sands approximately 6 ft below the bottom of the fill.

16. S-wave Test R3. Results from the surface shear wave refraction line R3 are not presented because the P-wave arrivals were so strong that the S-wave arrivals could not be accurately identified and analyzed.



KWAJ12
R1,R2

**** INPUT DATA ****

layer #	forward vel ft/s	forward t1 msec	reverse vel ft/s	reverse t1 msec
1	1329	0	1551	0
2	2892	3.7	2386	3.8
3	5095	7.1	5200	8.2
4	6217	9.7	6700	10.9

**** COMPUTED PROFILE ****

Ground Surface

1440

3 ______ 3.5

2610

7.5 ______ 9

5110

17.5 ______ 19.5

6400

This layer extends to unknown depth

NOTE: All depths in ft
All velocities in ft/s

Figure 8. Surface Seismic Refraction Test Results Lines R1 and R2

Crosshole and Downhole Surveys

17. S-wave. The results of the crosshole shear wave test are presented in figure 9. A plot of averaged velocities for all the tests versus depth shows a steady increase in velocity with depth except for an inversion zone between 8 and 21 ft (el 0 to el -13). This data set was divided into seven velocity zones which adequately defines this trend. The interpreted layers and velocities are presented in table 2. The downhole S-wave test results shown in figure 10, agree well with the crosshole tests. For the downhole test, the weakness of the shear waves propagating through the deeper material made the data difficult to analyze below a depth of 30 ft (el -22).

Table 2. Interpreted Layers and S-wave Velocities

Crosshole Shear Wave		Downhole Shear Wave	
Interface Depth (el), ft	Velocity fps	Interface Depth (el), ft	Velocity fps
	600		630
8 (0)		6 (2)	
	470		480
13 (-5)		11 (-3)	
	530		500
21 (-13)		19 (-11)	
	640		720
38 (-30)			
	740		
48 (-40)			
	840		
56 (-48)			
	940		

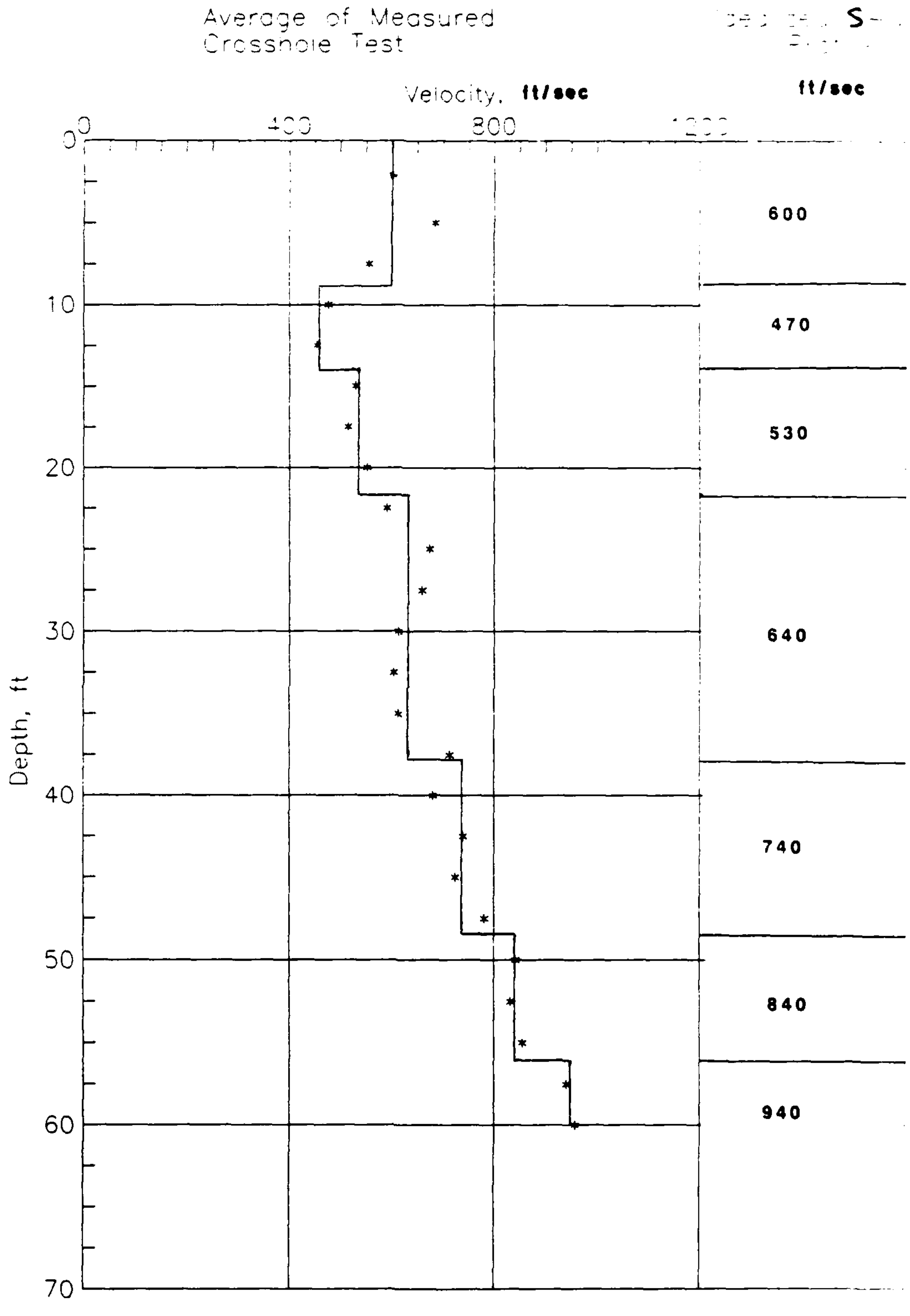


Figure 9. Crosshole Shear Wave Test Results

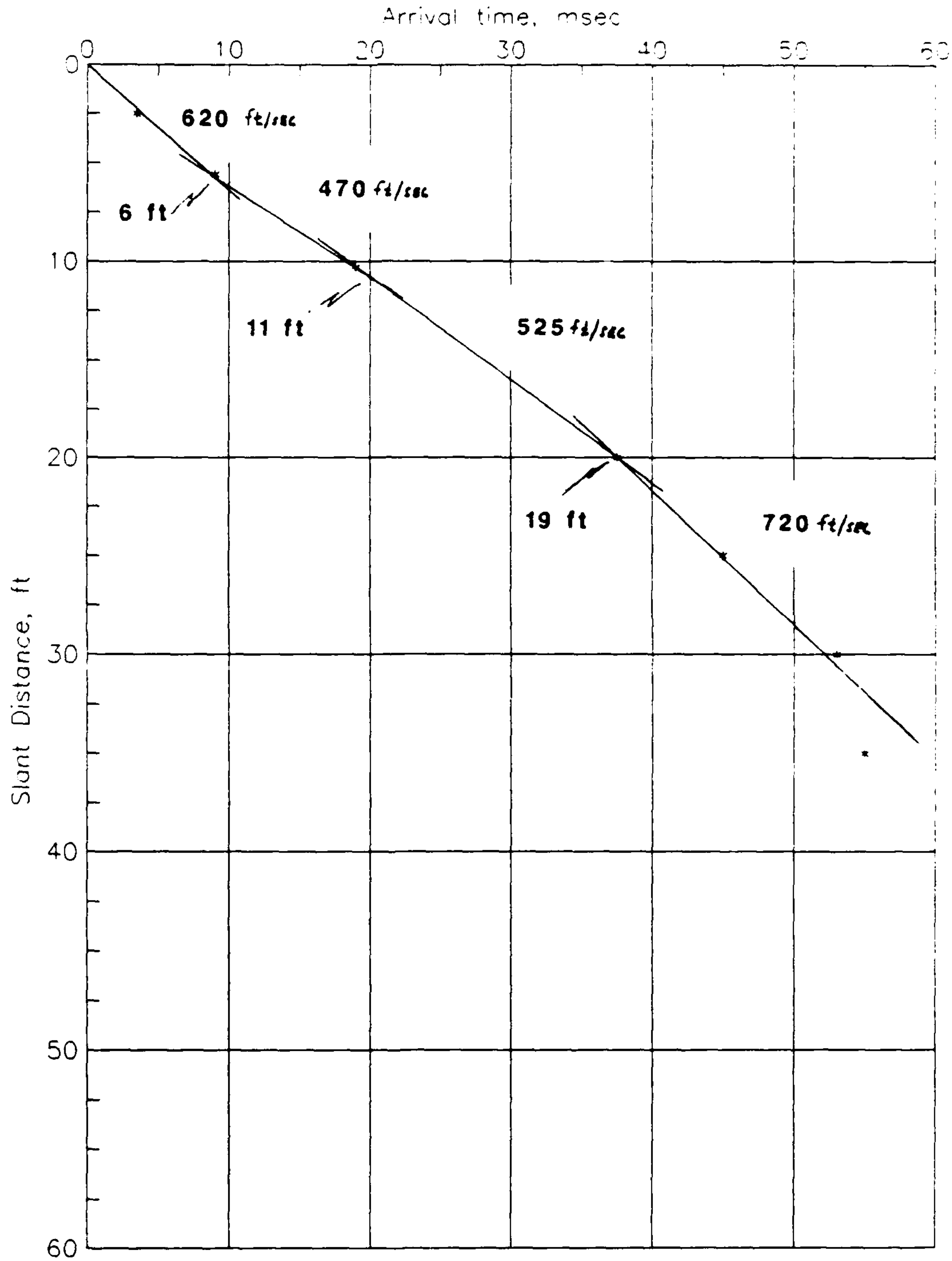


Figure 10. Downhole Shear Wave Test Results

18. P-wave. The results from the crosshole P-wave test indicate a five layer system, with the interpreted velocities and interfaces shown in table 3 and figure 11. Again, the velocities show a general increasing trend with increasing depth. The results of the downhole P-wave test are presented in figure 12. The downhole test shows an increasing velocity with depth profile which is also presented in table 3. The crosshole, downhole, and surface seismic tests show good agreement in the interpreted velocities and interfaces of the soil layers.

Table 3. Interpreted Layers and P-wave Velocities

Crosshole P-Wave Interface Depth (el), ft	Crosshole P-Wave Velocity fps	Downhole P-Wave Interface Depth (el), ft	Downhole P-Wave Velocity fps
	1500		1265
5 (3)		5 (3)	
	2150		3400
9 (-1)		8 (0)	
	5200		5000
15 (-7)		15 (-7)	
	6200		6280
20 (-12)			
	6600		
60 (-52)			

Interpretation

19. To make a meaningful interpretation of the results from the different tests into a generalized profile for the site, the data were first compared among the different tests at the same locations and then the

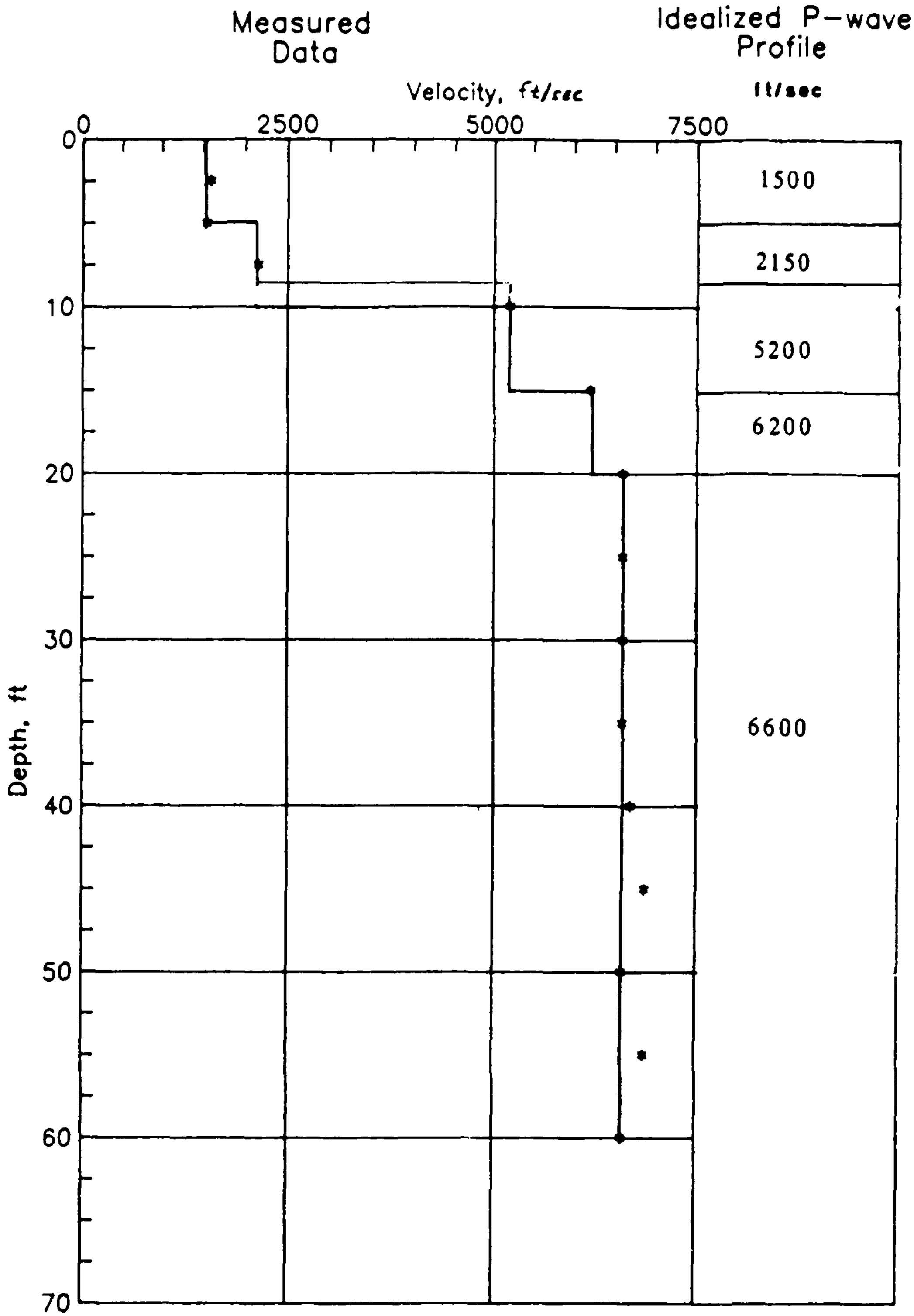


Figure 11. Crosshole Compression Wave Test Results

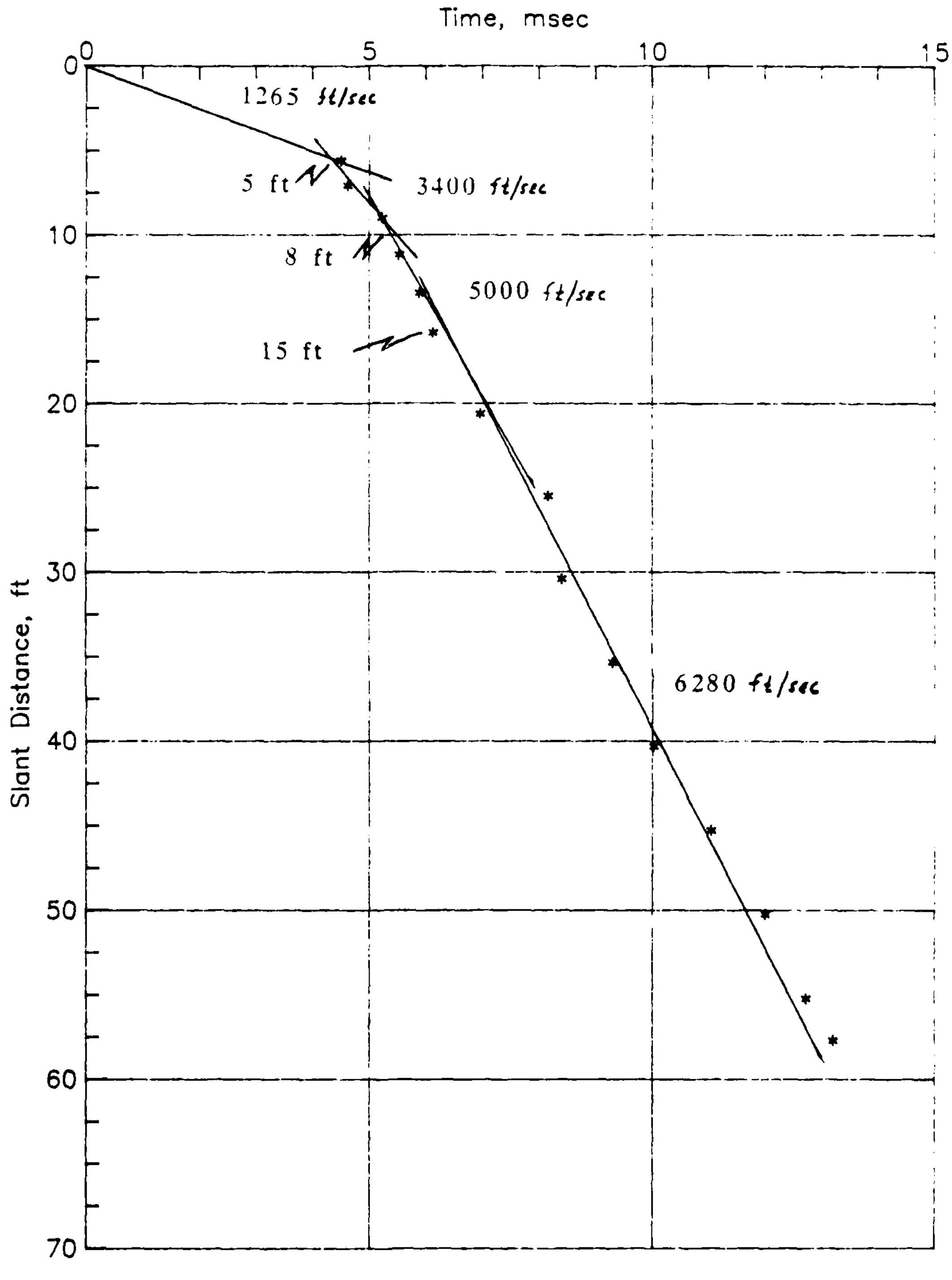


Figure 12. Composite Downhole Compression Wave Test Results

data sets were compiled into a composite profile. From these comparisons the results were analyzed and the idealized P- and S- wave velocity profiles were developed. These profiles are presented in figure 13 along with the four layer profile developed from the soil samples. The first layer of fill has a P-wave velocity range of 1,500 to 5,200 ft/sec with the 5,200 ft/sec interface being the top of the water table at a depth of 8 ft (el 0). This layer had a S-wave velocity range of 470 to 600 ft/sec. The fill showed an inversion in the S-wave velocity starting at the top of the water table. This inversion zone continues downward and includes layer two which is the beginning of the original soil deposit. However, this layer shows an increase in S-wave velocity to 530 ft/sec. This inversion zone was given a S-wave velocity of 510 ft/sec, weighted average of the 470 ft/sec and 530 ft/sec layers, for use in later soil modulus calculations. Layer 2 did not show an inversion in P-wave velocity which was measured to be 6,200 ft/sec because of the material being saturated. The third layer had a P-wave velocity of 6,600 ft/sec and a S-wave velocity of 640 ft/sec. The fourth layer had a P-wave velocity of 6,600 ft/sec and a range of S-wave velocities from 640 to 940 ft/sec. In general, the raw seismic data showed a some scatter which was also evidenced in the measured SPT values. These similar results suggest that the scatter in the data is mainly related to the soil nonhomogenity and therefore the velocities and properties reported are averages for the soil at depths tested with higher and lower actual values existing depending on test location.

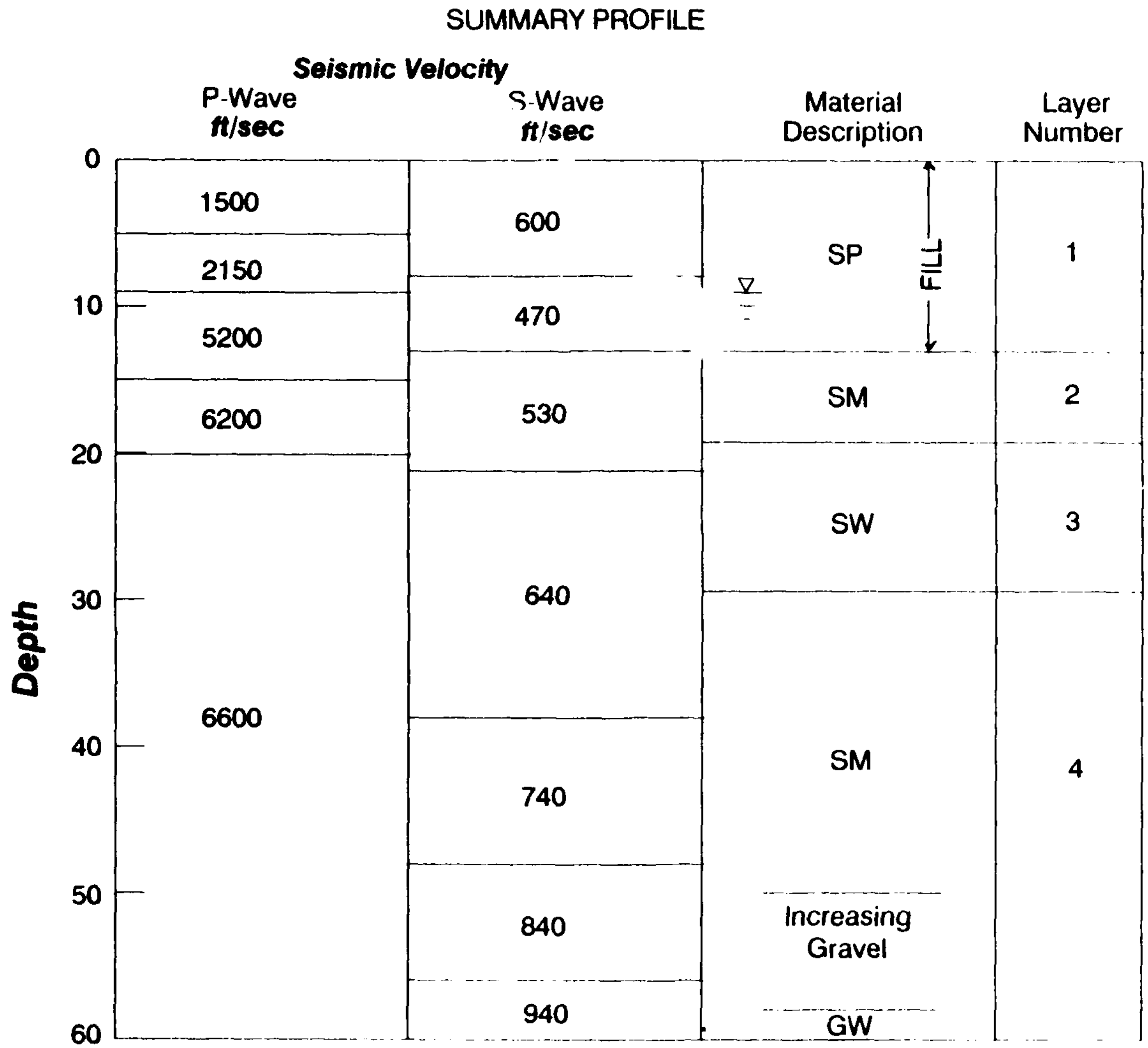


Figure 13. Composite Compression and Shear Wave Site Profile

Determination of Soil Material Parameters

20. <u>Determination of Shear Modulus.</u> Assuming an infinite, homogenous, isotropic, elastic medium, and solving the equations of wave motion for a shear wave, which is a wave confined to motion in a plane in the direction of propagation, results in Equation 1 (Richart et al, 1970).

$$V_S = \sqrt{G/\rho} \qquad (1)$$

where

$$\rho = \frac{\gamma_T}{g}$$

V_s = S-wave velocity ft/sec

G = shear modulus, lbs/ft^2

ρ = mass density of Soil, slugs/ft

γ_T = total unit weight of soil, lbs/ft^3

g = gravitational constant, ft/s^2

This equation states that S-wave velocity is dependent on the ratio of the shear modulus to the mass density of the medium. For this particular site a total unit weight of 100 lb/ft^3 was used for unsaturated soil and 120 lbs/ft^3

for saturated soil. The unit weight was converted to mass density; and, with the appropriate shear wave velocities substituted into Equation 1, the shear moduli, G, were computed. The results are tabulated in figure 14.

21. Determination of Poisson's ratio. Again, assuming an elastic medium, if the compression- and shear- wave velocities are known, Poisson's ratio, ν, can be determined from the ratio of these velocities, Equation 2. This relation is given below in Equation 3 (Department of the Army, 1967). The ratios were calculated using equation 2 and then used in expression 3 with the results of these calculations presented in figure 14.

$$V_R = V_P / V_S \qquad (2)$$

$$\nu = \frac{V_R^2 - 2}{2(V_R^2 - 1)} \qquad (3)$$

Vp = P-wave velocity, ft/sec

Vs = S-wave velocity, ft/sec

ν = Poisson's ratio

22. Determination of Young's and constrained moduli. Young's modulus, E, relates the stress to the resulting strain when a uniform stress is applied to plane sections of a body perpendicular to the applied force with the lateral surfaces free from constraint. The measurement of the velocity of this dilational (P-) wave through the body results in a "rod" velocity which with a value of the body's density, E can be determined directly (Kolsky, 1963).

SITE PARAMETERS

DEPTH ft	δ_T lb/ft^3	Vp ft/sec	Vs ft/sec	POISSON ratio	G lb/in^2	E lb/in^2	M lb/in^2	K2	K2 avg
2.5	100	1500	600	0.405	7764	21813	48525	89	
5.0	100	1500	600	0.405	7764	21813	48525	63	68
7.5	100	2150	600	0.458	7764	22636	99692	51	
10.0	120	5200	510	0.495	6731	20129	699792	38	
12.5	120	5200	510	0.495	6731	20129	699792	36	
15.0	120	5200	510	0.495	6731	20129	699792	34	33
17.5	120	6200	510	0.497	6731	20153	994820	32	
20.0	120	6200	510	0.497	6731	20153	994820	31	
22.5	120	6600	640	0.495	10600	31701	1127327	46	
25.0	120	6600	640	0.495	10600	31701	1127327	44	
27.5	120	6600	640	0.495	10600	31701	1127327	43	
30.0	120	6600	640	0.495	10600	31701	1127327	41	42
32.5	120	6600	640	0.495	10600	31701	1127327	40	
35.0	120	6600	640	0.495	10600	31701	1127327	39	
37.5	120	6600	640	0.495	10600	31701	1127327	38	
40.0	120	6600	750	0.493	14557	43482	1127330	50	
42.5	120	6600	750	0.493	14557	43482	1127330	49	
45.0	120	6600	750	0.493	14557	43482	1127330	48	48
47.5	120	6600	750	0.493	14557	43482	1127330	47	
50.0	120	6600	750	0.493	14557	43482	1127330	46	
52.5	120	6600	830	0.492	17829	53200	1127328	55	55
55.0	120	6600	830	0.492	17829	53200	1127328	54	
57.5	120	6600	925	0.490	22144	65987	1127330	66	65
60.0	120	6600	925	0.490	22144	65987	1127330	64	

Figure 14. Soil Moduli and Parameters

However, with in situ seismic testing the measured P-wave is related to the constrained modulus, M, and the body's density as a result of the different boundary conditions that exist for this type testing. The constrained modulus is so named because the lateral sides are constrained which more closely match the conditions present in an in situ seismic velocity measurement. Knowing either Young's modulus or the constrained modulus the other can be derived using the body's Poisson's ratio. In this study, the shear modulus and Poisson's ratio were determined, and then the soil's Young's modulus, E, was calculated using Equation 4. The constrained modulus, M, was determined using Equation 5. These calculated modulus values are shown in figure 14.

$$E = 2(1+\nu)G \qquad (4)$$

$$M = \frac{E(1+\nu)}{(1+\nu)(1-2\nu)} \qquad (5)$$

Procedure For Estimating Moduli at Different Confining Stresses

23. General. For engineering use it is necessary to determine soil moduli for the expected load magnitudes and loading conditions of the planned structure. These moduli can be arrived at using two approaches given that certain in situ properties are known. The first method involves selecting properties at a depth in which the overburden pressure is equal to the expected load of the structure (Department of Army, 1967). This method assumes that the soil is homogenous throughout the soil profile which encompasses the depth of the structure and the needed depth of soil to equal the structure's load. An alternative method allows an empirical constant, K2, to be determined for different layers in the profile and gives the advantage of using soil parameters that can vary in depth and therefore model the soil more precisely at the proper depth.

24. Determination of K2 Values. Investigators have shown that for sands shear modulus values are strongly influenced by the confining pressure, void ratio or relative density, and strain amplitude (Seed et al, 1970). For the case of low amplitude strains $< 5*(10^{-4})$, which is the range of in situ seismic wave velocity measurements, a Gmax is determined, and the resulting coefficient in equation 6, K2, is a K2max, which is also applicable to elastic response loading conditions. It should be further noted that these field

velocities are determined from total stress conditions which means that the moduli are for undrained loading conditions. The expression that relates Gmax to these factors is given in Equation 6.

$$G = 1000 K2 \sqrt{\sigma_m'} \qquad (6)$$

where

$$\sigma_m' = \frac{1}{3}(\sigma_1' + \sigma_2' + \sigma_3')$$
$$= \frac{1}{3}(\sigma_1' + 2K_0\sigma_1')$$
$$= 0.633\,\sigma_v'$$

Gmax = low strain amplitude shear modulus

K2 = empirical proportionality constant relating Gmax to $\sqrt{\sigma_m'}$

K_0 = coefficient of lateral pressure (assigned a value of 0.45)

σ_m' = effective mean confining stress, lbs/ft^2

σ_v' = effective vertical stress, lb/ft^2

It can be seen that the relational factor K2 can be determined from data at hand as a function of depth for the soil profile. These resulting K2's can be averaged within each layer with the resulting average K2 assigned to that zone of material. This assigned K2 can then be used to estimate shear moduli at selected confining stresses at any point in the profile by use of Equation 6. Table 4 given below is derived from data given in Seed et al. (1970) and can be used to estimate relative density for sands from K2 values.

Table 4.

Estimating Relative Density of Sands from K2 Values

Relative Density %		K2
Loose	30	34
Medium	40	40
	45	43
	60	52
Dense	75	61
Very Dense	90	70

25. **The calculated K2's for each depth increment in the soil profile are given in figure 14. A value of Ko=0.45 was assumed in determining the mean effective confining pressure.**

26. Soil Moduli Determinations. The use of the K2 parameter to estimate soil moduli for various loading conditions can be illustrated using this procedure to compare the moduli derived from the seismic and plate bearing tests. The moduli for the expected load was calculated using the sum of the static plus one-half of the cyclic plate bearing loads (3,140 lbs/ft^2) as the vertical effective stress. The in situ moduli from the seismic results were obtained from the same test elevation as the plate bearing test. These same calculations were repeated to a depth of 30 ft (el -22). A table of moduli values for this applied load to a depth of 30 ft and a comparison of Young's modulus between the seismic and plate bearing tests are presented in figure 15. The results from the seismic tests estimate a lower E than the plate bearing test results. The seismic results are strongly dependent on the magnitude of the S-wave velocity. The reported S-wave velocity of 600 ft/sec is a conservative average; however, if a velocity of 720 ft/sec is used, the agreement is much closer. This velocity is a raw average of all S-wave data at a depth of 2.5 ft. This result is also presented in figure 15 for comparison.

SOIL MODULI ESTIMATIONS

DEPTH ft	LOAD lb/ft^2	K2	Poisson's ratio	G* lb/in^2	E* lb/in^2
2.5	3140	89	0.410	27561	77722
5.0	3140	63	0.410	19510	55017
7.5	3140	51	0.470	15793	46433
10.0	3140	32	0.490	9910	29531
12.5	3140	36	0.490	11148	33222
15.0	3140	34	0.490	10529	31376
17.5	3140	32	0.490	9910	29531
20.0	3140	31	0.490	9600	28608
22.5	3140	46	0.490	14245	42450
25.0	3140	44	0.490	13626	40605
27.5	3140	43	0.490	13316	39682
30.0	3140	41	0.490	12697	37836

* Based on K2 soil parameter

COMPARISON OF MODULI DETERMINATIONS

Source	Load lb/ft^2	K2	Poisson's ratio	G lb/in^2	E lb/in^2	M lb/in^2
Seismic Test Procedure						
Vs=600 Vp=1500, ft/sec						
	3140	89	0.410	27561	77722	180678
Vs=720 Vp=1500, ft/sec						
	3140	128	0.350	39638	107023	171764
Plate Bearing Test					123000	

Figure 15. Comparison of Moduli Determinations from Seismic and Plate Bearing Test Methods

Conclusions

27. From the data that have been presented the following general conclusions are made.

a. The site can be divided into the following zones with material classifications and characteristic P- and S-wave velocities as follows:

Material	P-wave (ft/sec)	S-wave (ft/sec)
Layer 1: sand fill		
SP	1500[1] - 5200[2]	470 - 600
Layer 2: sand, silty		
SM	5200 - 6200	530
Layer 3: sand, fine		
SW	6600	640
Layer 4: sand, silty gravely		
SM	6600	640 - 940

[1]unsaturated [2]saturated

b. The SPT test results show that the measured N-values have considerable scatter especially in the upper 10 ft at the site. The scatter is probably due to the naturally occurring gravel and cobbles that were in layer one and buried construction materials in the near surface. The average equivalent $(N1)_{60}$ blowcount for the fill is 25 with the original sands between 14 and 29 ft in depth (el -6 to -21) having a value of 21. The sands below a depth of 29 ft (el -21) show a decrease in the $(N1)_{60}$ blowcount to a value of 15. A global site value for the sands would be 22 which corresponds to a medium dense sand.

c. The Shear and Young's moduli and Poisson's ratio have been determined for each test depth and tabulated in figure 14. These parameters have been grouped according to idealized layers and presented in figure 16. Throughout the entire profile the shear modulus ranges from $7.7\times(10^3)$ lbs/in^2 at the surface to $22\times(10^3)$ lbs/in^2 at a depth of 60 ft. Young's modulus varies from $22\times(10^3)$ lbs/in^2 at the surface to $66\times(10^3)$ lbs/in^2 at the bottom of the profile. The unsaturated materials near the surface have a Poisson's ratio of 0.40 with the saturated materials below the water table having an expected value of 0.49. There exists a zone between 10 and 20 ft in depth (el -2 to -12) where the moduli decrease due to the inversion in the S-wave velocity profile at these depths.

d. A K2 parameter was calculated for each test depth and an average K2 assigned to zones in the profile and shown in figures 14 and 16. Layer one has a K2 of 68 in the upper 7 ft and 33 for the remainder. The second layer also has a K2 value of 33. The upper 2 feet of layer three has a K2 of 33 which increases to 42 below that depth. The fourth layer ranges from a K2 of 42 at a depth of 29 ft (el -21) to a K2 of 65 at a depth of 60 ft (el -52).

SITE PARAMETERS

LAYER	DEPTH ft	γ_T lb/ft³	SEISMIC VELOCITY Vp ft/sec	Vs ft/sec	ν	G lb/in²	E lb/in²	SOIL PARMAMETERS K2	K2 avg	Dr	(N1)60	Dr
1	2.5	100	1500	600	0.405	7764	21813	89	68	88	25	67
	5.0				0.405			63				
	7.5		2150		0.458		22636	51				
	10.0	120	5200	510	0.495	6731	20129	38				
	12.5							36				
2	15.0							34	33	30	21	64
	17.5		6200		0.497		20153	32				
3	20.0							31			21	64
	22.5		6600	640	0.495	10600	31701	46				
	25.0							44				
	27.5							43				
4	30.0							41	42	43	15	55
	32.5							40				
	35.0							39				
	37.5							38				
	40.0			750	0.493	14557	43482	50				
	42.5							49				
	45.0							48	48	55		
	47.5							47				
	50.0							46				
	52.5			830	0.492	17829	53200	55	55	65		
	55.0							54				
	57.5			925	0.490	22144	65987	66	65	80		
---	60.0	120	6600	925	0.490	22144	65987	64				

Figure 16. Summary of Soil Moduli and Parameters

e. The K2 parameter was used to estimate moduli for a load of 3,140 lbs/ft^2 and then compared with the plate bearing test results. The plate bearing test estimates an E of 123,000 lbs/in^2 which is higher than the E of 78,000 lbs/ft^2 derived from the K2 parameter.

f. Comparing the relative densities of the sands at the site using correlations based on calculated K2 values and $(N1)_{60}$ blowcounts shows good agreement except that the K2 derived relative density estimates are lower for the sands between the 10 and 30 ft depth range (el -2 to - 22). These relative density estimates are shown in figure 16.

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APPENDIX A

BORING LOGS

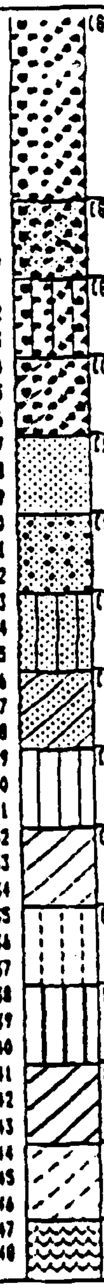
B-1
B-2
B-3

Project Location: AND TEST DATA

Project Number: LEGEND
Boring Number: DOC

PHI Degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or >CR<	Feet
TRIAL TEST: ANGLE OF INTERNAL FRICTION	TRIAL TEST: COHESION	SIEVE TEST	SIEVE TEST	SIEVE TEST	SPECIFIC GRAVITY TEST	MOIST UNIT WEIGHT	PLASTICITY INDEX	NATURAL MOISTURE CONTENT	BLOWS PER FOOT OR CORE RECOVERY >CR<	

* SIGNIFIES NUMBER OF BLOWS ON 3" OD RING SAMPLER IN LIEU OF STANDARD 2" OD PENETRATION SAMPLER ——→*



Visual Classification

Feet	Visual Classification
1–7	(GP) POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
8–10	(GW) WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
11–13	(GM) SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
14–16	(GC) CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
17–19	(SP) POORLY-GRADED SANDS GRAVELLY SANDS, LITTLE OR NO FINES
20–22	(SW) WELL-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
23–25	(SM) SILTY SANDS, SAND-SILT MIXTURES
26–28	(SC) CLAYEY SANDS, SAND-CLAY MIXTURES
29–31	(ML) INORGANIC SILTS AND VERY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
32–34	(CL) INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
35–37	(OL) ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
38–40	(MH) INORGANIC SILTS, ELASTIC SILTS
41–43	(CH) INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
44–46	(OH) ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
47–48	(PT) PEAT AND OTHER HIGHLY ORGANIC SOILS

Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): NOT MEASURED
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

Project Number: KW0190
Boring Number: B-1
Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 8.5' MSL
North: 3,165,699
East: 1,694,278
Completion Date: 20 OCT 1989

PHI degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or > CR (	Feet	Visual Classification
									19	1	(SP) 0-13.0' SAND, POORLY GRADED, SOME ZONES OF WELL GRADED AND SILTY SANDS, SCATTERED COBBLES AND GRAVEL, MEDIUM DENSE TO DENSE, TAN TO PINK, MOIST (FILL) (GRADING TO WET AT 7.0')
										2	
						115		11	*83	3	
									41	4	
										5	
									*12	6	
								23	20	7	
										8	
									16	9	
									14	10	
										11	
										12	
										13	(SM) 13.0-19.0' SAND, SILTY, VERY FINE GRAINED, WITH GRAVEL, GRADING TO GRAY, WET
										14	
									17	15	
										16	
										17	
										18	
										19	(SW) 19.0-29.0' SAND, WELL GRADED, SOME GRAVEL AND SILT, MEDIUM DENSE, TAN TO WHITE, WET
									29	20	
										21	
										22	
										23	
										24	
									21	25	
										26	
										27	
										28	
										29	(SM) 29.0-60.5' SAND, SILTY, WITH GRAVEL, DENSE TO MEDIUM DENSE, TAN TO WHITE, WET
									53	30	
										31	
										32	
										33	
										34	
									19	35	
										36	
										37	
										38	
										39	
									16	40	
										41	
										42	

Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): NOT MEASURED
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

Project Number: KW0190
Boring Number: B-1
Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 8.5' MSL
North: 3,165,699
East: 1,694,278
Completion Date: 20 OCT 1989

PHI Degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or > CR <	Feet	Visual Classification
										43	SAND, SILTY, WITH GRAVEL,
										44	DENSE TO MEDIUM DENSE,
									22	45	TAN TO WHITE, WET
										46	
										47	
										48	
										49	
									20	50	
										51	
										52	
										53	
										54	
										55	(GRADING TO MORE GRAVELS
										56	AT 55.0')
										57	
										58	
										59	
									20	60	

Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): 9.0
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

Project Number: KW0190
Boring Number: B-2
Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 8.5' MSL
North: 3,165,690
East: 1,694,282
Completion Date: 24 OCT 1989

PHI degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCE)	PI (%)	Wn (%)	N or > CR <	Feet	Visual Classification
		5	90	5	2.821	94		9	*28	1	[SP] 0-14.0' SAND, POORLY GRADED,
										2	ZONES OF WELL GRADED AND
		23	70	7	2.768	102		11	*33	3	SILTY SANDS. SCATTERED
		21	75	4				10	*13	4	COBBLES, MEDIUM DENSE, TAN TO
										5	PINK, MOIST (FILL)
		6	89	5				12	*7	6	
		6	90	4	2.788	93		21	*28	7	(GRADING TO WET AT 7.0')
										8	
		1	96	3					*24	9	
		9	84	7	2.804				18	10	
										11	
										12	
										13	
										14	[SM] 14.0-19.0' SAND, SILTY,
		20	32	48					16	15	VERY FINE GRAINED, W/ GRAVEL,
										16	MEDIUM DENSE, GRADING TO
										17	GRAY, WET
										18	
										19	[SW] 19.0-29.0' SAND, WELL GRADED,
		23	69	9					19	20	W/ GRAVEL, SOME SILT,
										21	MEDIUM DENSE, TAN TO WHITE,
										22	WET
										23	
										24	
		21	72	7	2.785				20	25	
										26	
										27	
										28	
										29	[SM] 29.0-57.0' SAND, SILTY,
		45	39	16					9	30	W/ GRAVEL, ZONES OF MORE
										31	GRAVELS, LOOSE TO MEDIUM
										32	DENSE, TAN TO WHITE, WET
										33	
										34	
		38	43	19					12	35	
										36	
										37	
										38	
										39	
		26	56	18					13	40	
										41	
										42	

Project Name: GBR-X
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): 9.0
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

Project Number: KW0190
Boring Number: B-2
Project Name: GBR-X
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 8.5' MSL
North: 3,165,690
East: 1,694,282
Completion Date: 24 OCT 1989

PHI Degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or CR	Feet	Visual Classification
										43	
										44	
		14	67	19	2.854				22	45	
										46	
										47	
										48	
										49	(GRADING TO MORE GRAVEL AT 50.0')
		26	57	17					25	50	
										51	
										52	
										53	
										54	
										55	
										56	
										57	(GW) 57.0-60.5' GRAVEL, SANDY, MEDIUM DENSE, TAN, WET
										58	
										59	
		68	27	5					15	60	

Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): 8.0
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

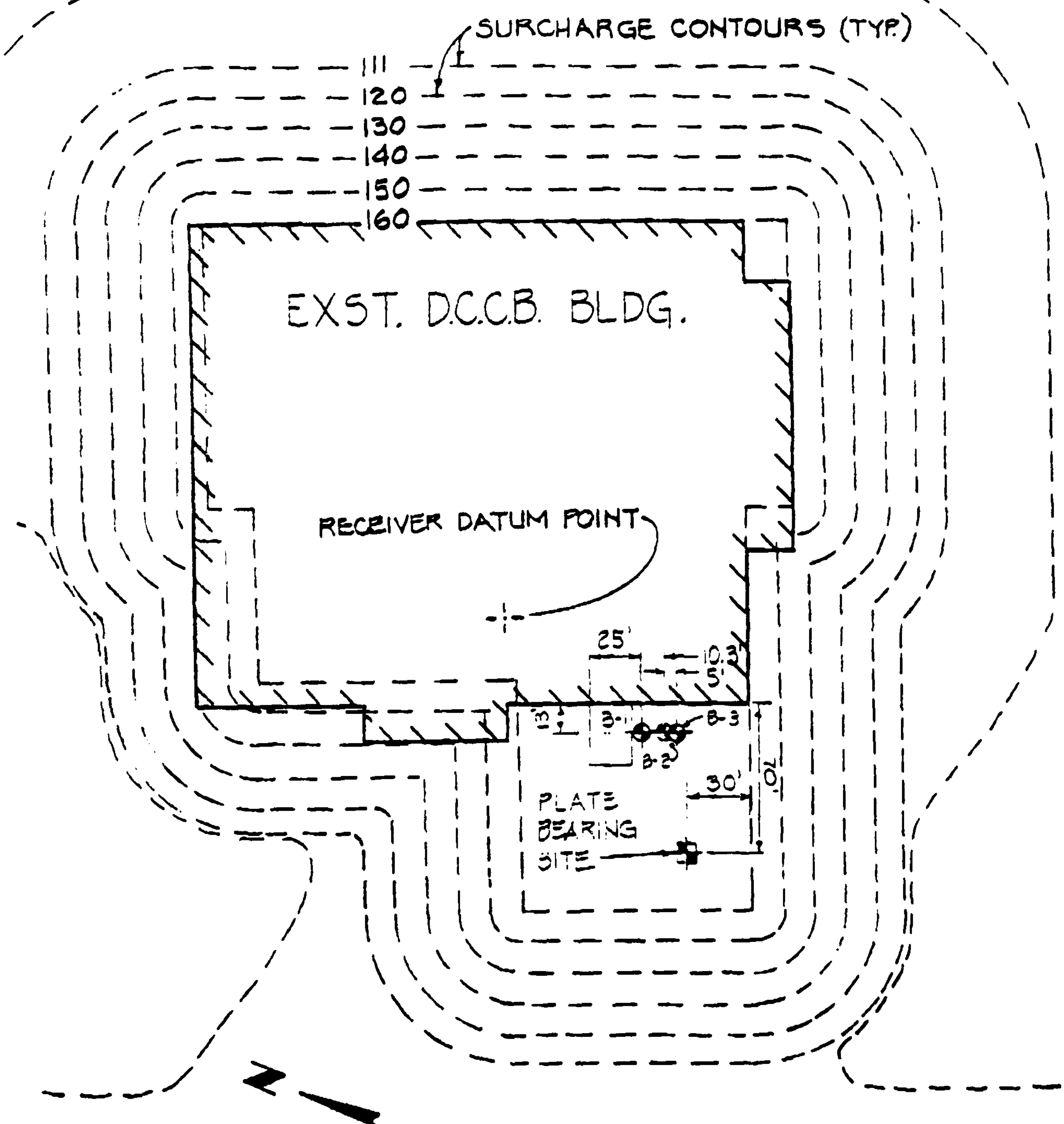
Project Number: KW0190
Boring Number: B-3
Project Name: GBR-I
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 9.5' MSL
North: 3,165,687
East: 1,694,284
Completion Date: 26 OCT 1989

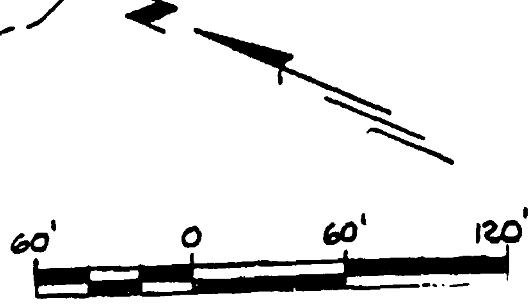
PHI Degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or > CR <	Feet	Visual Classification
									20	1	(SP) 0-14.0' SAND, POORLY GRADED, ZONES OF WELL GRADED AND SILTY SANDS, SCATTERED COBBLES AND GRAVEL, MEDIUM DENSE, TAN TO PINK, MOIST (LOOSE AT 5.0') (GRADING TO WET AT 7.0') (FILL)
										2	
									24	3	
									16	4	
									8	5	
										6	
										7	
										8	
										9	
									15	10	
										11	
										12	
										13	
										14	(SM) 14.0-19.0' SAND, SILTY, VERY FINE GRAINED, WITH GRAVEL, MEDIUM DENSE, GRADING TO GRAY, WET
										15	
										16	
										17	
										18	
										19	(SW) 19.0-29.0' SAND, WELL GRADED, WITH GRAVEL, SOME SILT, MEDIUM DENSE, TAN TO WHITE, WET
									17	20	
										21	
										22	
										23	
										24	
										25	
										26	
										27	
										28	
										29	(SM) 29.0-60.5' SAND, SILTY, WITH GRAVEL, ZONES OF MORE GRAVEL, MEDIUM DENSE TO LOOSE, TAN TO WHITE, WET
									21	30	
										31	
										32	
										33	
										34	
										35	
										36	
										37	
										38	
										39	
									11	40	
										41	
										42	

Project Name: GBR-X
Project Location: KWAJALEIN ISLAND
Depth to Water (ft): 8.0
Drill Company: FAR EAST DISTRICT
Drill Rig: CME-55
Inspector: OKADA
Casing Depth (ft): NA
Core Recovery (%): NA

Project Number: KW0190
Boring Number: B-3
Project Name: GBR-X
Project Location: KWAJALEIN ISLAND
Top of Hole (elev): 8.5' MSL
North: 3,165,687
East: 1,694,284
Completion Date: 26 OCT 1989

PHI degrees	C (TSF)	GRAV (%)	SAND (%)	FINE (%)	Gs	γ_m (PCF)	PI (%)	Wn (%)	N or > CR <	Feet	Visual Classification
										43	
										44	
										45	
										46	
										47	
										48	
										49	(GRADING TO GRAVEL, SANDY,
									27	50	WITH SILT)
										51	
										52	
										53	
										54	
										55	
										56	
										57	
										58	
										59	
									22	60	





FY89 MCD PN9219150
GROUND BASED RADAR COMPLEX
KWAJALEIN, USAKA
BORING LOCATION PLAN
DEC 1989 PLATE

APPENDIX B

PLATE BEARING TEST

(ER 105-1-5)

FROM (Name)	OFFICE SYMBOL	TELEPHONE NO.	RELEASER'S SIGNATURE		
G. MASATSUGU	CEPOD-ED-GS	(808) 438-8875/1941	JOHN S. RAVINA, CH, FM&S BR, POD		
TO (Name)	OFFICE SYMBOL	TELEPHONE NO.	# PAGES	PRECEDENCE	DTG
DON YULE	CEWES	(601) 634-2235	8	ROUTINE	18Dec89

SUBJECT

*U.S. Government Printing Office 1986-623-350

ENG FORM 6000 1 FEB 73

U.S. ARMY ENGINEER DIVISION, PACIFIC OCEAN
CORPS OF ENGINEERS

PROJECT TITLE GBR-X SH NO. 1 OF 3 SHS
LOCATION ADJ TO DCCB, KWAJALEIN ISL SECTION
DRAWING(S) NO.
COMPUTED BY OTO DATE 11/14 CHECKED BY DATE 11/17/89

DESIGN ANALYSIS

STATIC PLATE BEARING TEST WORKSHEET

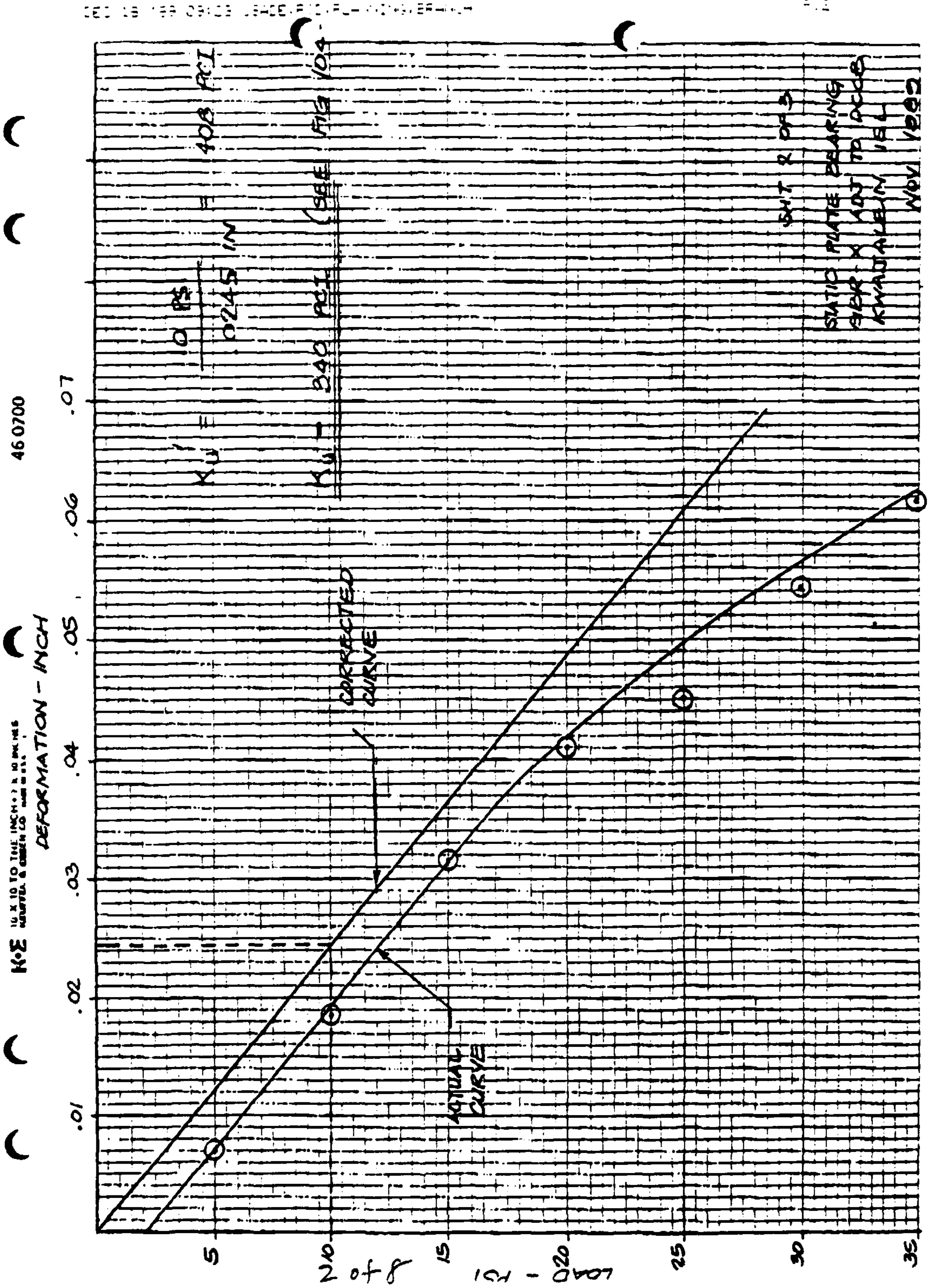
LOAD (KG)	LOAD (PSI)	DIAL READINGS #1	#2	#3	DEFLECTIONS (IN) #1	#2	#3	AVG
400	1.2	.0088	.0018	.0051	← ZERO READING			
1600	5.0	.0177	.0091	.0100	.0089	.0073	.0049	.0070
3200	10.0	.0324	.0142	.0244	.0236	.0124	.0193	.0184
4800	15.0	.0413	.0336	.0345	.0333	.0318	.0294	.0315
6400	20.0	.0485	.0444	.0459	.0397	.0426	.0408	.0410
8000	25.0	.0516	.0489	.0502	.0428	.0471	.0451	.0450
9600	29.9	.0605	.0570	.0607	.0517	.0552	.0556	.0542
11,200	34.9	.0678	.0645	.0689	.0590	.0627	.0638	.0618

K_u = 340 pci

MIL STD 621A
METHOD 104
MODULUS OF SOIL REACTION

1 of 8

DEC 18 '99 09:03 USACE-RIC-PL-H-ICHW-BR-H-UH



408

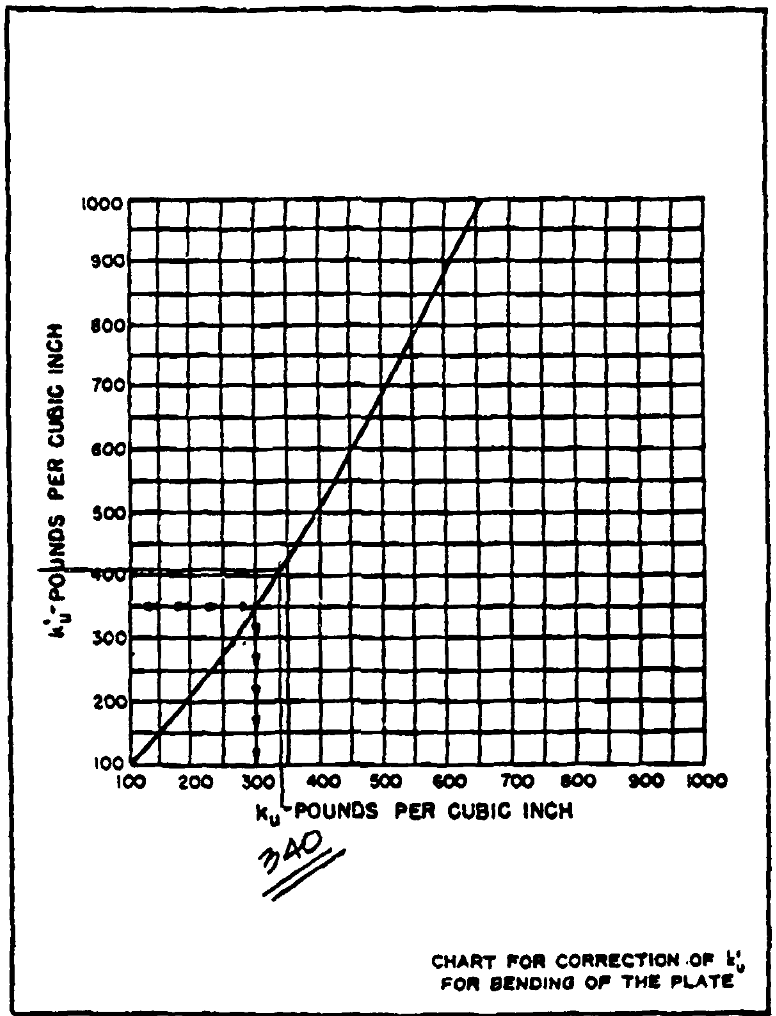


FIGURE 104-3. Correction of k'_u for bending of the plate.

SHT 3 OF 3
STATIC PLATE BEARING
GBR-X ADJ TO DCCB
KWAJALEIN ISL
NOV 89
3 of 8

UNIT WEIGHT DETERMINATION "VOLUME OF HOLE" METHODS	DATE 25 OCT 1989

PROJECT	TEST SITE	SAMPLE NUMBER
GBR - X	ADJACENT TO PLATE BEARING TEST SITE	

ADDITIONAL SPECIFICATIONS

CONVERSION FACTORS

1 in. = 2.54 cm. 1 gm./cc. or 62.4 lb./cu. ft. = Unit weight of water

1 lb. = 453.6 gm. 1 cu. ft. = 1728 cu. in.

CALIBRATION OF STANDARD MATERIAL	STANDARD MATERIAL (Check one) ☒ SAND ☐ OIL ☐ OTHER (Specify)				
	UNITS				
APPARATUS OR TARE NUMBER					
1. WEIGHT OF APPARATUS OR TARE FILLED					
2. WEIGHT OF APPARATUS OR TARE EMPTY					
3. WEIGHT OF MATERIAL (1.-2.)					
4. VOLUME OF APPARATUS OR TARE					
5. UNIT WEIGHT OF MATERIAL (3/4)					
6. AVERAGE UNIT WEIGHT OF MATERIAL	LB./CU. FT.	93.0	93.0		

CALIBRATION OF APPARATUS		TEMPLATE NUMBER A		CONE NUMBER A	
	UNITS				
7. INITIAL WEIGHT OF APPARATUS + SAND	LBS				
8. FINAL WEIGHT OF APPARATUS + SAND VOL =	.0408 FT	^{3} x 93	PCF	=	
9. WEIGHT OF SAND IN TEMPLATE AND/OR CONE	LBS	3.79	3.79		

"VOLUME OF HOLE"					
	UNITS				
10. INITIAL WEIGHT OF APPARATUS + MATERIAL	LBS	15.42	16.43		
11. FINAL WEIGHT OF APPARATUS + MATERIAL	LBS	8.69	9.29		
12. WEIGHT OF MATERIAL RELEASED (10.-11.)	LBS	6.73	7.14		
13. WEIGHT OF MATERIAL IN HOLE (For oil, same as 12. For sand, 12.-9.)	LBS	2.94	3.35		
14. VOLUME OF HOLE (13/6)	FT^3	.0316	.0360		

DD FORM AUG 57 1215

4 of 8

WATER CONTENT DETERMINATION

	UNITS						
TARE NUMBER		88	55				
15. WEIGHT WET SOIL & TARE	GMS	317.5	304.3				
16. WEIGHT DRY SOIL & TARE	GMS	296.7	280.9				
17. WEIGHT WATER (15.-16.)	GMS	20.8	23.4				
18. WEIGHT TARE	GMS	65.2	66.1				
19. WEIGHT DRY SOIL (16.-18.)	GMS	231.5	214.8				
20. WATER CONTENT ($\frac{17}{19}$. x 100)	%	9.0	10.9				
21. AVERAGE WATER CONTENT	PERCENT						

UNIT WEIGHT DETERMINATION

	UNITS			
TARE NUMBER		RW-2	C	
22. WEIGHT WET SOIL & TARE	LBS	4.64	4.97	
23. WEIGHT TARE	LBS	0.71	0.71	
24. WEIGHT WET SOIL (22.-23.)	LBS	3.93	4.26	
25. WET UNIT WEIGHT (24./14.)	LB./CU. FT.	124.4	118.3	
26. DRY UNIT WEIGHT (25. x $\frac{100}{100+21.}$)	LB./CU. FT.	114.1	106.7	

REMARKS

TESTS TAKEN 12" BELOW EXIST'G GRADE (APPROX ELEV 7' MSL).

TEST PIT LOG

0.0" ———— GROUND SURFACE

GRAY/TAN SAND (COMPACTED) (SP)

4.0" ————

TAN SAND (FILL) WITH CORAL GRAVEL & OCCASSIONAL COBBLES (SP)

12.0" ————

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
OTO		

U. S. GOVERNMENT PRINTING OFFICE, 1967

578

U.S. ARMY ENGINEER DIVISION, PACIFIC OCEAN
CORPS OF ENGINEERS

PROJECT TITLE GBR-X SH NO. 1 OF 3 SHS
LOCATION ADJ TO DCCB, KWAJALEIN ISL SECTION
DRAWING(S) NO.
COMPUTED BY OTO DATE 11/15 CHECKED BY [illegible] DATE 12/15/89

DESIGN ANALYSIS

DYNAMIC PLATE BEARING

LOAD (KG)	DIAL READINGS #1	#2	#3	DEFORMATION (IN) #1	#2	#3	AVG (IN)
0	.0171	.0051	.0066	← "ZERO" READING			
6200	.0466	.0180	.0181	.0295	.0129	.0115	.01797
6200	.0467	.0193	.0186	.0296	.0142	.0120	.01860
7800	.0473	.0195	.0193	.0302	.0144	.0127	.01910
#11 6200	.0469	.0195	.0188	.0298	.0144	.0122	.01880
7800	.0476	.0198	.0196	.0305	.0147	.0130	.01940
#12 6200	.0470	.0197	.0190	.0299	.0146	.0124	.01897
7800	.0475	.0198	.0194	.0304	.0147	.0128	.01930
#13 6200	.0474	.0198	.0194	.0303	.0147	.0128	.01927
7800	.0475	.0198	.0195	.0304	.0147	.0129	.01933
#14 * 6200	.0465	.0186	.0184	.0294	.0135	.0118	.01823
7800	.0475	.0198	.0196	.0304	.0147	.0130	.01937
#15 6200	.0470	.0198	.0192	.0299	.0147	.0126	.01907

* UNLOAD WAS INCORRECT — SEE GRAPH

•• PLOT LOAD VS. AVG DEFORMATION

7800 KG = 3503 PSF ≈ 3500 } 720 = Δl
6200 KG = 2783 PSF ≈ 2780

* 4900 KG = 2200 PSF — JACK SLIPPED TO 4900 KG INSTEAD OF 6200

POD Form 115
1 Jul 70

U.S. ARMY ENGINEER DIVISION, PACIFIC OCEAN
CORPS OF ENGINEERS

PROJECT TITLE GBR-X SH NO. 2 OF 3 SHS
LOCATION ADJ TO DCCB, KWAJALEIN ISL SECTION
DRAWING(S) NO.
COMPUTED BY OTO DATE 11/15 CHECKED BY [illegible] DATE [illegible]

DESIGN ANALYSIS

CALCULATE AVG REBOUND
OVER LAST FIVE CYCLES

CYCLE	LOAD	UNLOAD	REBOUND (IN)
11	.01910	.01880	.00030
12	.01940	.01897	.00043
13	.01930	.01927	.00003
* 14	.01933	.01873 * ~~.01823~~	.00060 * ~~.00110~~
15	.01937	.01907	.00030

* CORRECTION MADE FROM GRAPH AVG REBOUND = .00033 ~~.00043~~

FROM EC 1110-345-147, 11 JULY 1966

$$E' = 25.5 \frac{\Delta l}{S} (1 - \mu^2) \quad \text{where } \mu = 0.3$$

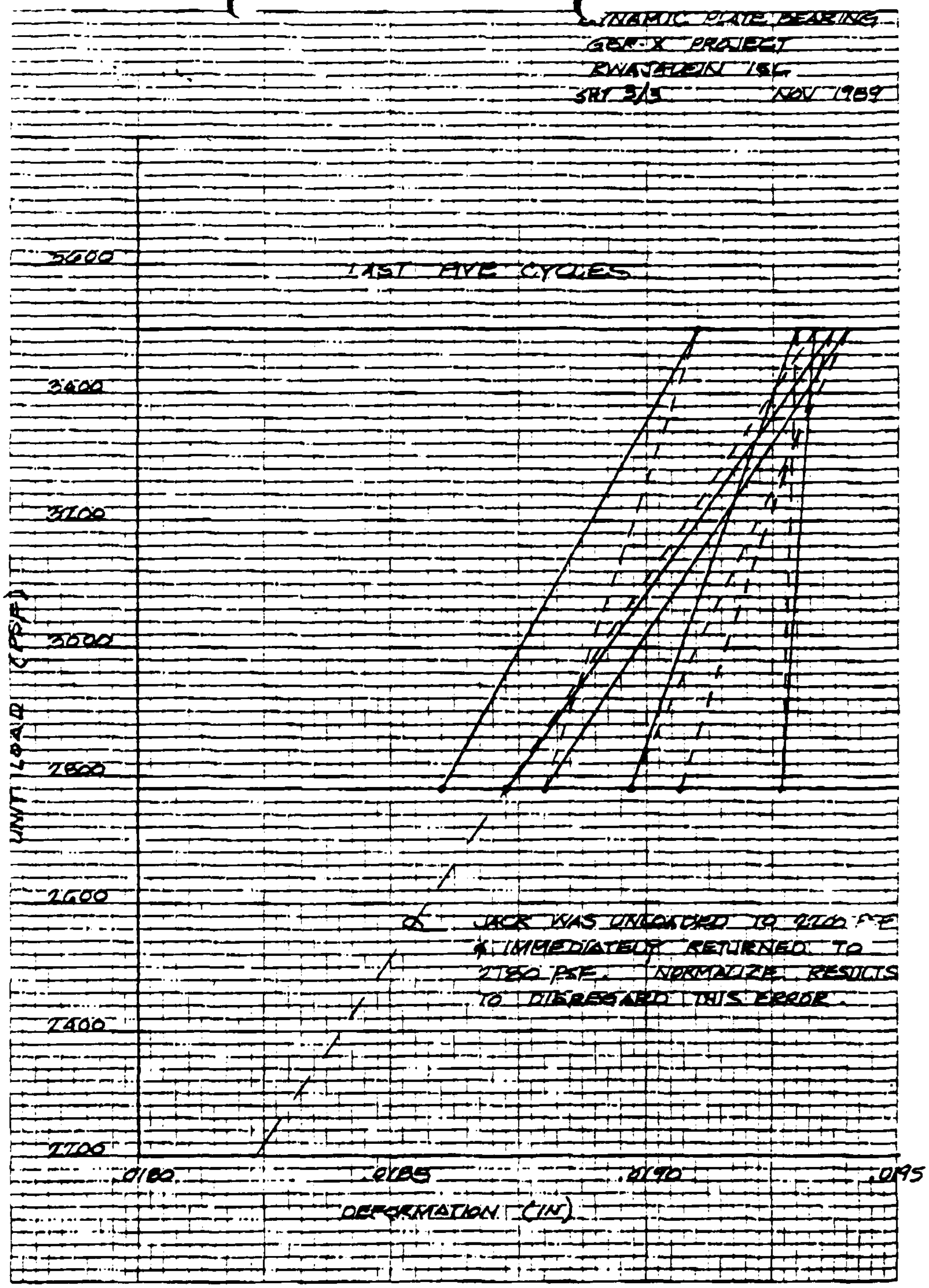
$$E' = 23.2 \frac{\Delta l}{S} \qquad \Delta l = 720 \text{ PSF} \qquad S = .00033 \text{ IN}$$

$$= \frac{23.2\,(720)}{\cancel{.00043}\ .00033}$$

$E' = 50.6 \times 10^6$ PSF (UNCORRECTED MODULUS)

$E = 17.8 \times 10^6$ PSF (CORRECTED MODULUS)

POD Form 115
1 Jul 70


DYNAMIC PLATE BEARING
GBR-X PROJECT
KWAJALEIN ISL
SHT 3/3
NOV 1989
LAST FIVE CYCLES
3600
3400
3200
3000
2800
2600
2400
2200
UNIT LOAD (PSF)
.0180
.0185
.0190
.0195
DEFORMATION (IN)
JACK WAS UNLOADED TO 2200 PSF
& IMMEDIATELY RETURNED TO
2780 PSF. NORMALIZE RESULTS
TO DISREGARD THIS ERROR.

8/8